TETRA TECH

Report of Geotechnical Investigation
Milltown State Park

Milltown, Montana

March 1, 2012

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March 1, 2012

Mr. Christopher Anderson
DJ&A, P.C. Engineers. Planners. Surveyors
3202 Russel Street
Missoula, MT 59801

SUBJECT: Geotechnical Investigation
Milltown State Park
Milltown, Montana
Tetra Tech Project No. 114-570451

Dear Mr. Anderson:

We have performed a geotechnical investigation of the soil conditions at the proposed location of the Milltown State Park in Milltown, Montana.

This report presents our investigation, the results of our findings, and our foundation and slope analyses. It is important that we provide consultation and review during the design phase, and field observation and testing services during construction, to ensure complete implementation of the geotechnical design recommendations.

If you have any questions, please contact myself or Jeremy Dierking at 406-543-3045.

Respectfully submitted,

TETRA TECH

Marco Fellin, P.E.
Project Geotechnical Engineer

Enclosure: 4 copies

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Report of Geotechnical Investigation

Milltown State Park
Milltown, Montana

Prepared for:

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PURPOSE AND SCOPE OF STUDY

Per Tetra Tech's January 10, 2012 contract with DJ&A, P.C, and the 50% design Scope of Services obtained from DJ&A, the scope of Tetra Tech's services included; "using available as-built data in concert with field investigation data obtained on site, prepare a Memorandum summarizing the field investigation results, slope stability analyses, shoring foundation design at the railroad bridge, and pavement design recommendations (including bituminous surface treatment options) for the access roadway and parking areas." Per an October 27, 2011 e-mail from Tetra Tech to DJ&A, Tetra Tech's scope of services for the I-90 Bridge West Slope and MRL Bridge location would also include the following:

I-90 West Slope: *Tetra Tech will obtain available as-built slope construction and subsurface soils data from MDT, the Army Corp of Engineers, CH2MHill, or other involved parties. The as-built data will be utilized by Tetra Tech to perform a slope stability analysis for the slope in its current state prior to trail construction, and obtain an existing Factor of Safety. Tetra Tech will utilize a 100-year flood event in the analysis. Following a decision by the design team regarding a preferred wall system to utilize along the trail at this location, Tetra Tech will perform a second slope stability analysis to determine the change in slope Factor of Safety due to the addition of the wall weight and trail system to the slope.*

MRL Bridge: *Tetra Tech will drill two soil borings to depths on the order of 30 to 40 feet at the toe of slope at the proposed sheet-pile or h-pile wall location. If access is extremely difficult on one side of the embankment or other, Tetra Tech may opt to drill only one soil boring. The soils information obtained will be utilized to design the embedment depth required for the pile shoring system adjacent to the railroad. Tetra Tech will also use a track-hoe to excavate one or two vertical trenches into the existing embankment slope (one on either side) to identify the soil types in the existing embankment that will need to be retained by the permanent shoring system. Tetra Tech will also provide design soil pressures for the wall system, backfill material required, and drainage recommendations.*

Samples obtained during the field investigation were tested in Tetra Tech's laboratory to determine the physical and engineering characteristics of the on-site soils. Due to the amount of geotechnical field data and analyses performed for this project, Tetra Tech utilized a report format in lieu of a memo to present the data. This report summarizes the field data and presents conclusions and recommendations for design and construction of the proposed pavilion, retaining systems, trails, and parking area based on the proposed construction and subsurface conditions encountered. The report also includes design parameters and geotechnical engineering considerations related to construction.

PROPOSED CONSTRUCTION

Milltown State Park is located at the confluence of the Blackfoot and Clark Fork Rivers on the outskirts of Milltown, Montana. The proposed project consists of the following site improvements; walking trails, parking areas, pavilion, and retaining structures at the undercrossing of the I-90 and Montana Rail Link bridges. We understand that some of the site features may not be constructed due to funding, but we have included recommendations in this report should they eventually be constructed.

Specific building loading information was not available at the time of report preparation. If building structures are to be constructed in the future, structural loads should be provided to Tetra Tech to evaluate our geotechnical recommendations. The trails, roads, and parking areas are anticipated to be surfaced with asphalt concrete.

FIELD EXPLORATION

The field exploration was conducted on January 10, 2012. Five borings were drilled within the proposed areas of the trails, road, parking, and undercrossings to obtain information on subsurface conditions and one test pit was excavated near the proposed location of the Montana Rail Link retaining structures (Drawing No. 570451-1). Locations of the exploration borings were marked in the field by Tetra Tech personnel, based on the site map provided by DJ&A. Elevations for DH-5 and the test pit were surveyed by Tetra Tech using the existing northwest MRL wingwall as a benchmark. The elevation for the wingwall was subsequently obtained from HDR. Borings were advanced through the overburden soils with a truck-mounted drill rig equipped with 8-¼-inch diameter hollow-stem augers and were logged by Tetra Tech's field engineer. The test pit was excavated with a steel-tracked excavator and logged by Tetra Tech's field engineer.

Samples of the subsurface materials were obtained with 2-inch outside-diameter split-spoon samplers driven into the various strata using a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler each of three successive 6-inch increments was recorded; the total number of blows required to advance the sampler the second and third 6-inch increments is the penetration resistance (N value). This is the standard penetration test described by ASTM Method D1586. Penetration resistance values indicate the relative density or consistency of the soils. Bulk samples of soil were obtained from the hollow-stem augers and test pit spoils at select locations. Depths at which the samples were obtained and the penetration resistance values are shown on the logs of exploration borings.

LABORATORY TESTING

Samples obtained during the field exploration were taken to Tetra Tech's laboratory, where they were observed and visually classified in accordance with ASTM Method D2487, which is based on the Unified Soil Classification System. Representative samples were selected for testing to determine the physical properties of the soils in general accordance with ASTM or other approved procedures.

Tests Conducted:	To Determine:
Grain-size Distribution	Size and distribution of soil particles; that is, clay, silt, sand and gravel.
Atterberg Limits	The effect of varying water content on the consistency of fine-grained soils.
Natural Moisture Content	Moisture content representative of field conditions at the time samples were taken.

Moisture-Density Relationship	The optimum moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
Resistivity and pH	The combination of these characteristics determines the potential of soil to corrode metal.
Sulfate Content	Potential of soils to deteriorate normal strength concrete.

Field and laboratory test results are summarized on Figures 7 through 13 in the Appendix. These data, along with the field information, were used to prepare the exploratory boring and test pit logs in Figures 1 through 6.

SITE CONDITIONS

Existing site conditions consist of rolling topography near the proposed pavilion site (Photo 1), with gravel road surfacing on the existing road and parking area (Photo 2). The proposed trail along the Blackfoot River will be cut into the existing slopes at the bridge undercrossings (Photo 3). On-site vegetation consists of landscaped grass, trees, and shrubs, natural grass, weeds, and deciduous forest outside of the developed areas.

Photo 1. Looking Northeast at drill rig setup on BH-3, the proposed pavilion location.

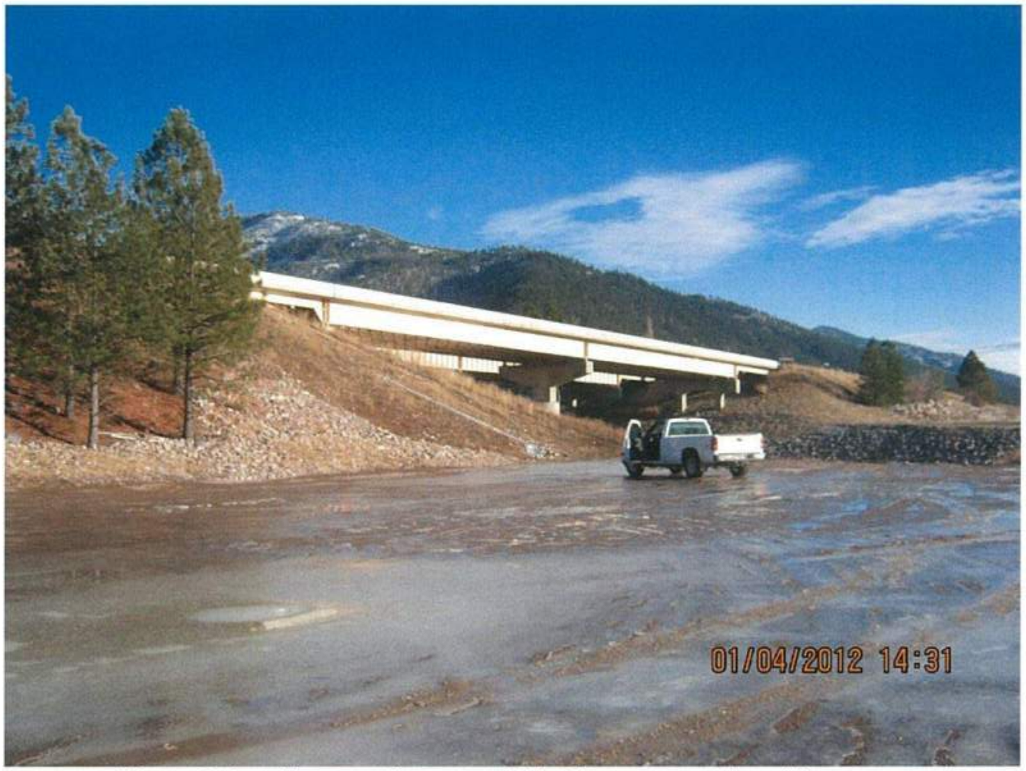

Photo 2. Looking northeast at BH-4 in proposed parking area.


Photo 3. MRL undercrossing slope and drill rig at BH-5.

National Seismic Hazard Maps prepared by the USGS 2008 depict probabilistic strong ground motions and spectral accelerations with 10, 5 and 2 percent probabilities of exceedance in any 50 year period for the conterminous United States. NEHRP (Recommended Provisions for Seismic Regulations for New Buildings and Other Structures) design criteria are based on a 10 percent probability of exceedance, or in other words, a 90 percent probability of not being exceeded in a 50 year period. Based on the USGS National Seismic Hazard Mapping 2008 data base, the peak ground acceleration in Milltown, Montana having a 10 percent probability of exceedance in any 50 year period is 0.09g. The USGS database presents spectral response acceleration data in bedrock for short (0.2 sec) periods (S_A) and for long (1 sec) periods (S_1) for similar probability and 50 year return periods. According to USGS design procedures, these acceleration data are then adjusted upward or amplified depending on soil classification to reflect magnification effects as the earthquake wave energies pass from bedrock into soil. The values are then reduced by a factor that accounts for partial damping of the wave energy by the structure. The final values obtained (known as S_{DS} and S_{D1}) become the basis for the structural design and in this case at the proposed site are estimated as 0.504 g (S_{DS}) and 0.239 g (S_{D1}).

The methods of ASCE/SEI 7-05 require that the properties of the soil at the proposed building site be classified as one of several site classes. The seismic design parameters for this site include a seismic zone soil profile type of (D), in accordance with the above referenced standard. Site Class D corresponds to a stiff soil profile with an undrained shear strength between 1,000 and 2,000 psf and standard penetration resistance values between 15 and 50 blows per foot. We have based this classification on the laboratory test data, exploration boring information, and our experience in the vicinity of this project.

SUBSURFACE CONDITIONS

The subsoils encountered during the field investigation generally consisted of gravel with varying percentages of silt and sand and occasional discontinuous layers of silt. The boring logs should be referenced for complete descriptions of the soil types and their estimated depths. A characterization of the subsurface profile normally includes grouping soils with similar physical and engineering properties into a number of distinct layers. The representative subsurface layers at the site are presented below, starting at the ground surface.

TOPSOIL/FILL

Topsoil or fill was encountered at the surface of borings BH-3 and BH-5 and at the test pit location, and extended to depths of 0.5 to 1 foot. The topsoil is generally brown to dark brown in color and consists of silt and sand. Samples of the topsoil obtained at the surface had natural moisture contents ranging from 2 to 5 percent.

Fill was encountered in the test pit (railroad embankment) to the maximum depth explored (approximately 10 feet). The fill classified as silty gravel and sand with the occasional cobbles and boulders, Figure 9.

SAND and GRAVEL

Natural sand and gravel were encountered in all of the borings below the surface, topsoil, or fill, and extended beyond the maximum depth explored (40.5 feet). The sand and gravel contain varying percentages of silt and clay, and occasional cobbles and boulders throughout the depth of the borings. Penetration resistance values in the sand and gravel ranged between 22 to greater than 50 blows per foot, which is indicative of medium dense to very dense soil stratum. The natural moisture content for samples obtained in the gravel ranged from 2 to 9 percent. Classifications for samples tested in the laboratory included; silty sand with gravel, silty clayey sand with gravel, and silty gravel with sand (Figures 7, 8, and 9). Liquid and plastic limit tests performed on three samples ranged from non-plastic to a liquid limit of 20 and a plasticity index of 4. Based on our experience with similar deposits, we anticipate the sand and gravel will exhibit moderate to high shear strength characteristics and be slightly compressible under normal loading conditions.

SILT

Natural silt was encountered below the sand and gravel in borings BH-2, BH-4, and BH-5, at depths on the order of 7 feet, extending to depths on the order of 22 feet. Penetration resistance values in the silt ranged between 9 to 14 blows per foot, which is indicative of stiff to very stiff soil stratum. Natural moisture contents in the silt layer ranged from 2 to 5 percent.

GROUNDWATER

Subsurface water was encountered in BH-5 at a depth of approximately 28 feet at the time of the field exploration (January 10, 2012). Groundwater elevations typically fluctuate depending on the season, local irrigation practices, and the amount of precipitation during a particular year. Numerous factors contribute to groundwater fluctuations, and evaluation of such factors is beyond the scope of this report.

ENGINEERING ANALYSIS AND RECOMMENDATIONS

EXCAVATIONS

It is critical that trail work in the vicinity of the I-90 and MRL Bridges and slopes does not cause construction-induced damage to the existing foundations or slopes. We recommend the following precautions or procedures be incorporated into the project plans. We also recommend that the project manager ensure or a representative of the geotechnical engineer be on site full-time to ensure these items are implemented during the construction process;

1. Grading for the trail section on the I-90 and MRL Bridge slopes should be accomplished using equipment no larger than a tracked 'Bobcat-size' grader, which will minimize the ground pressures imposed on the slope.
2. Gabion baskets should be fabricated and filled in place, and not transported to their final location. Rock to fill the gabion baskets should be hauled to the wall locations using a Bobcat-sized front end loader.
3. Soil, cobbles, or boulders cut from the existing slopes cannot be piled on the existing slope, they must either be hauled off-site or used to backfill behind the gabion walls to construct the trail system. Piling excavated soils on the existing slope will create a concentrated pressure zone and increase the potential for slope failure.
4. Use no larger than a Bobcat-sized front end loader to place the gravel and asphalt trail structure.
5. Compact the gravel and asphalt layers for the trail under the I-90 and MRL structures using equipment no larger than a smaller remote-controlled static steel roller.

SITE GRADING

We anticipate that the majority of site grading for this project will be for the trail system along the Blackfoot River, as well as boat ramps or trails leading down to the river from the new park area. The access road and main parking area are planned to be constructed approximately at existing grade.

Because a limited number of geotechnical borings were drilled for the project, materials samples must be obtained and tested during construction to ensure the soils are properly moisture conditioned and compacted prior to constructing structures, trails, parking areas, or roads.

Design and construction criteria presented below should be observed for site preparation purposes and when preparing project documents.

1. All topsoil, debris, and other deleterious material should be removed from the proposed construction locations.

2. All native gravel, fill, and backfill for walls should be approved by the geotechnical engineer, moisture-conditioned to within 2 percent of optimum moisture content, and placed in uniform lifts of suitable thickness for the compaction equipment. It should then be compacted to the following minimum dry densities as determined by ASTM D698 or to the minimum percentage of the relative density determined by the combination of ASTM D4253 and D4254, whichever method is applicable for the material being compacted.

	ASTM D698	ASTM D4253& D4254
Below Foundations and Floor Slabs	98%	75%
Below Gabion Baskets	98%	75%
Wall Backfill	95%	70%
Subgrade and Base for Paved Areas	95%	70%
Exterior Foundation Walls	95%	70%
Overlot Fill	90%	65%

3. Imported or on-site granular material used as engineered gravel fill should meet the following grading limits and be compacted in accordance with item 2 above.

Sieve or Screen Size	Percent Passing
3-inch	100
No. 4	25 – 60
No. 200	0 – 12

4. The natural sand and gravel are suitable for use as engineered gravel fill below structure footings, floor slabs, paved areas, and as over-lot fill, provided it meets the requirements of item 3 above for engineered gravel fill, and is moisture conditioned and compacted in accordance with item 2 above. The on-site silt material should not be

5. The contractor is responsible for providing safe working conditions in connection with underground excavations. Temporary construction excavations which workers will enter will be governed by OSHA guideline 1926.6542, Appendix B to subpart P. For planning purposes, subsoils encountered in the exploration borings classify as Type C.

6. Site grading must be developed and maintained during and after construction to rapidly drain surface and roof run-off well away from foundation and subgrade soils.

BUILDING FOUNDATIONS

Natural sand and gravel were encountered at conventional footing depths at the location of the proposed pavilion. We recommend conventional spread footings bearing on compacted natural sand and gravel. Our calculations indicate continuous spread footings bearing on the natural sand and gravel can be proportioned for an allowable bearing pressure of 4,000 psf. Based on the theory of elasticity and the anticipated structural loads, and using a bearing pressure of 4,000 psf, we estimate the total settlement for spread footings supported on the natural sand and gravel to be approximately1-inch or less, which is within the tolerable limit for the type of construction proposed.

The lateral resistance of spread footings is controlled by a combination of sliding resistance between the footing and the foundation materials and passive earth pressure against the side of the footing. Criteria for calculating the lateral resistance are presented below. The following design and construction criteria should be observed for a conventional spread footing foundation. The construction details should be considered when preparing the project documents.

1. Structure footings should be supported on natural sand and gravel and designed for an allowable bearing pressure of 4,000 psf, provided settlements as outlined above are acceptable. ***When the structures are designed, Tetra Tech should be provided the structural loads so the recommendations in this report can be evaluated.***

2. Exterior footings should be placed at least 42 inches below grade for frost protection.

3. The minimum width of column footings should be at least 24 inches and at least 16 inches for continuous spread footings, or in accordance with applicable building codes, whichever is more restrictive.

4. Footing lateral loads may be resisted by friction between the footing base and supporting soil, and lateral bearing pressure against the sides of footings and grade beams. For design purposes, a friction coefficient of 0.45 for natural sand and gravel or engineered gravel fill, and a lateral bearing pressure of 400 psf per foot of depth for the natural sand and gravel are appropriate.

5. A representative of Tetra Tech's geotechnical engineer should observe all footing excavations and test the compaction of all native sand and gravel or engineered fill prior to placement of concrete forms.

6. Concrete in contact with the soil should be designed using Type I-II cement.

FLOOR SLABS

Performance of slab-on-grade construction is dependent on having uniform subgrade support beneath the slab. Floor slabs should be supported on the natural sand and gravel or engineered gravel fill placed to raise site contours. It is also customary to provide a gravel leveling course beneath floor slabs. This is normally a construction convenience rather than a structural requirement.

The following recommendations should be considered for concrete slab-on-grade construction.

1. A minimum 4-inch thick layer of free-draining gravel should be placed between the slabs and the natural gravel or engineered gravel fill as a leveling course. This material should consist of minus 3/4-inch aggregate with less than 60 percent passing the No. 4 sieve and less than 10 passing the No. 200 sieve.

2. To reduce the effects of differential movement, floor slabs should be separated from all bearing walls and columns with expansion joints, which allow unrestrained vertical movement. Floor slab control joints should be used to reduce damage due to shrinkage cracking. The requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.

3. Concrete floor slabs supported on native sand and gravel or engineered gravel fill as described above should be designed using a modulus of subgrade reaction of 400 pounds per square inch (psi) per inch.

PAVEMENT SECTIONS – ACCESS ROAD AND PARKING AREAS

We understand that the trail system will be paved, and that DJ&A will provide a standard pavement design section for all of the trails systems associated with this project. Per our Scope of Work, Tetra Tech will provide pavement recommendations for the access road and parking area.

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and the traffic loadings. A uniformly compacted subgrade is vital for good pavement performance. Traffic along the proposed access road and main parking area is expected to be light, consisting of passenger cars, pickup trucks, and occasional buses transporting tourists and floaters. It is anticipated the pavement can be divided into one category of traffic intensity, equal to one equivalent single-axle load per day or less.

Pavement design procedures are based on strength properties of the subgrade and pavement materials, along with the design traffic conditions. For pavement thickness design, soils are represented by means of a California Bearing Ratio (CBR) value. A sample of the silty sand with gravel was obtained from boring DH-2, and laboratory testing indicates a CBR value on the order of 9 percent, which is considered a medium strength subgrade soil for supporting pavements under controlled placement conditions. The pavement thickness design was developed using the methods presented in the AASHTO *Guide for Design of Pavement Structures*, 1993.

We understand that cost-savings on this project are critical, therefore Tetra Tech will present several pavement section alternatives here for evaluation;

Option 1) Traditional asphalt concrete pavement with crushed granular base course. For this option, we recommend the following flexible pavement sections or an approved equivalent. Guideline specifications for construction and materials selection, based on the Montana Department of Transportation Standard Specifications for Road and Bridge Construction, are included in the Appendix. As an alternative, the Montana Public Works guideline specifications may also be used.

Material	(Thickness, Inches)
Asphalt Concrete Surfacing (MDT Grade B)	2*
¾" or 1 ½" Crushed Aggregate Base Course (MDT Type B, Grade 2)	6
Total	8

*Tetra Tech typically does not typically recommend less than 3 inches of asphalt surfacing for paved parking or roadway sections, primarily as a precaution to prevent premature distress in the pavement section due to traffic loading, turning movements, and environmental factors. Tetra Tech suggests that 2 inches of asphalt may very well provide a durable riding surface for quite a few years, but has the potential to show pavement distress and cracking sooner than if a 3-inch pavement section were used.

Option 2) Stabilized Base Layer with Surface Seal. This option includes the following steps: Grade existing gravel road section and compact, place 3 inches of 1-inch minus crushed base course gravel, inject or spray 'Base One' stabilizer into the 3-inch base course layer, grade to distribute Base One throughout the 3-inch layer, allow to cure for 10 days, then seal surface with 'Otta Seal' followed by a chip seal if necessary. This construction procedure has been used with success on many low volume roads in Minnesota and surrounding states, provides a sound pavement structure, a surface that is similar to asphalt, and is very economical. A recent 26-foot wide roadway was re-constructed in Minnesota using this procedure at a cost of approximately $45,000 per mile, compared to the estimated cost of $250,000 per mile using the traditional base course and 2-inch asphalt pavement section. If the design team choses to utilize this method of construction, Tetra Tech can assist in developing the project specifications and special provision. The following section should be used;

Material	(Thickness, Inches)
Chip Seal (optional)	0.5
Otta Seal (similar to slurry seal, specifications to be provided by Tetra Tech should this option be used).	1.5 (placed in two ¾ inch lifts)
¾" Crushed Aggregate Base Course (MDT Type B, Grade 2) treated with 0.0075 Gallons per square yard per inch of depth of Base One stabilizer	3
Compacted Existing Road and Parking Area Surface	N.A.
Total	4.5 to 5.0

It should be noted that the riding surface utilizing Option 2 will be more susceptible to distress than Option 1, and may not be as smooth as a traditional asphalt pavement, but the structure itself is designed for 15 years. When the riding surface begins to show distress, Tetra Tech suggests that additional seal coats can be applied as necessary.

RETAINING WALLS

Per a Preliminary Report prepared by HDR Engineering, Inc. (HDR) for the Clark Fork River Trail Bridge Undercrossings, and our recent discussions with HDR, several retaining structures will be constructed beneath and adjacent to the I-90 Bridge and MRL Railroad Bridge structures. Gabion walls, one to two rows high, will be constructed on either side of the trail beneath both the I-90 and MRL Bridges on the west slopes. The gabions will be designed by HDR. Two soldier-pile walls will be constructed adjacent to the MRL Bridge west abutment to support the cut required for the trail to pass beneath the bridge. HDR will design the soldier-pile wall. Following are our geotechnical recommendations for each wall type.

Gabions: I-90 Bridges and MRL Bridge West Slopes

Gabion retaining walls will be subjected to horizontal loading due to lateral earth pressure. The lateral earth pressure is a function of the natural and backfill soil types and acceptable wall movements, which affect soil strain and mobilize the shear strength of the soil. Soil movement is required to develop greater internal shear strength and lower the lateral pressure on the wall. Distribution of the lateral earth pressures on the structure depends on soil type and wall movements or deflection. In most cases, a triangular pressure distribution is satisfactory for design and is usually represented as an at-rest equivalent fluid unit weight or pressure.

Due to the permeability of the site soils and the proposed gabion baskets, we do not recommend a drainage system behind the walls. The gabion baskets should be supported on the native site soils or gravel fill that have been compacted as discussed in *Site Grading*. Backfill should be placed in uniform lifts and compacted as described in the *Site Grading* section of this report. Care should be taken as not to over-compact the backfill since this could induce excessive lateral stresses on the walls.

The design and construction criteria presented below should be observed for retaining walls.

1. Gabion baskets should be placed on compacted native soils and designed for an allowable bearing pressure of 3,000 psf.
2. Retaining walls that are laterally supported and expected to undergo only a slight amount of deflection should be designed for a lateral earth pressure computed on the basis of an at-rest equivalent fluid unit weight of 55 pcf for backfill consisting of free-draining sand and gravel material. We recommend passive earth pressure be computed on the basis of an equivalent fluid unit weight of 400 pcf for backfill consisting of the free-draining sand and gravel material. Conventional safety factors used in structural analysis for items such as overturning moments and sliding should be used in the design.
3. ***IT IS VERY IMPORTANT THAT ONLY STATIC LIGHT-DUTY COMPACTION EQUIPMENT BE USED TO COMPACT THE BACKFILL FOR THE WALLS AND TRAIL PAVEMENT SECTION TO PREVENT UNDUE STRESSES ON THE EXISTING SLOPES.***

Soldier-Pile Walls: MRL Bridge

Two soldier-pile retaining walls will be constructed adjacent to the existing MRL west bridge abutment. The south wall will be approximately 6 to 16 feet in height (highest adjacent to the existing abutment), and will be approximately 36 feet in length. The north wall will be approximately 6 to 12 feet in height, and approximately 30 feet in length. Per discussions with HDR, the current design includes W18x119, HP14x89, and HP12x53 pile sections extending on the order of 12 to 30 feet below the bottom of the proposed wall, depending on the wall height. A 30-inch auger hole will be drilled to allow vertical placement of the W18 sections, and backfilled.

The soldier-pile wall will be subject to horizontal loading due to lateral earth pressure as well as train loading. Per discussions with HDR, we understand the soldier-pile wall design will be based on active earth pressure theory and a flexible wall state, with a maximum allowable deflection on the order of 1.5 inches at the top of new wall.

The design and construction criteria presented below should be observed for design of the soldier pile retaining wall.

1. For retained soils above approximate elevation 3,262 to 3,264 feet, a friction angle of 32 degrees and a total unit weight of 135 pcf should be used for design.
2. For embedded piles below the trail elevation, a friction angle of 28 degrees and a total unit weight of 120 pcf should be used.
3. A groundwater elevation of 3,240 feet should be used for design.
4. Because the lack of cohesion in the existing fill material, and the anticipated vibrations from train traffic, we recommend that the holes for pile placement be excavated using hollow-stem auger drilling methods, which will essentially provide temporary casing for the hole and allow for placement of fill prior to removing the augers. Montana Rail Link is

concerned about the stability of the existing fill slopes, and has indicated that vibratory methods of installation or drilling (air-rotary) are not allowed on this project.

5. The contractor should anticipate cobbles and also boulders up to 18 inches in size or larger during the augering process. Should the augering method not be suitable to remove the boulders, the contractor should be prepared to use other methods for removal, which could include equipment or bars to drop into the hole to break apart the boulders. As indicated in Item 4, vibratory or air-rotary methods to complete the excavations or install the piles are not allowed. If removal of the auger is required to remove obstructions, the contractor should be prepared to temporarily case the hole while removing the obstruction(s).
6. Prior to removing the augers, a Controlled Density Material (CDM) must be tremied to the bottom of the auger and placed to the top of wall elevation. The augers must then be removed, ensuring the head of CDM remains above the bottom of auger.
7. Following removal of the augers, the soldier beam shall be placed and centered in the excavation, and be fully encased in CDM.
8. The CDM must be allowed to set up for the amount of time specified in the contract. Following setup, the contractor must excavate a zone of soil and CDM to allow for placement of the precast retaining panels. The excavation should extend approximately 1 foot behind the back face of the wall to allow for placement of pea gravel.
9. We recommend including drainage provisions behind the wall, consisting of placing a layer of 'Seperation-Stabilization' geotextile (See MDT Standard Specification Section 716 – Geotextiles) along the entire back face of the excavation, then placing a minimum 1-foot width of pea gravel in the entire gap behind the wall face. The pea gravel layer should be connected to a perforated drain pipe at the base of the wall that daylights towards the river.

SLOPE STABILITY ANALYSES

Slope stability analyses were performed using the computer program SLIDE (version 6.013), developed by Rocscience, Inc., to determine the factor of safety of critical slip surfaces using vertical slice limit equilibrium methods. The SLIDE program can perform grid searches of circular slip centers and non-circular block searches to generate failure surfaces to identify the critical slip surfaces and determine the factor of safety for the critical failure surfaces based upon the input criteria. The input includes the slope profile, internal material configurations, soil strength properties, and location of the phreatic surface within the profile. The random circular failure surfaces were analyzed using the Simplified Bishop and Janbu Method of Slices.

I-90 Bridges West Slope

A slope stability analysis was performed to evaluate the impact of the new trail system on the stability of the existing west slope below the I-90 bridges. We also performed analyses to evaluate the impact of the new trail system on the stability of the slopes outside of the bridge footprint. The slope geometries assumed in the stability analysis model were based on available design documents obtained from CH2MHill for the Milltown Bridge Infrastructure Mitigation project. Both micropiles and a grout 'curtain' were previously constructed during the project in an attempt to stabilize the west slope. Because the limits and quality of the grout curtain are uncertain, Tetra Tech did not include the grout curtain in the slope analyses.

The existing slope topography, riprap geometry, and micropile locations and geometry were based on the cross-sections shown on CH2MHill's Erosion Protection plan sheet Nos. 5 and 6, dated August 29, 2008. The subsurface geometry, engineering properties of subsurface materials, and micropile shear strengths were based on the cross-sections and slope stability analysis models included in CM2MHill's micropile design calculations, dated August 2, 2007. Blackfoot River water surface elevations were based on Table 3-4 from CH2MHill's geotechnical report dated December 2006. The proposed trail configuration was based on Figure 9 of HDR's Preliminary Report dated September 30, 2011. Soil parameters used in the stability analyses are presented below in Table 1.

Table 1. Summary of Soil Parameters, Stability Analyses

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (degrees)
Embankment Fill	135	0	35
Sediment – Sand	108	0	26
Sediment – Clay	100	500	0
Alluvium	130	0	38
Bedrock	145	10000	35
Riprap	120	0	40
Gabion Fill	130	100	40

The stability analysis included the following assumptions:

1. Per the CH2MHill report, the existing slope was modeled at a consistent slope angle of 2H:1V. The existing bridge piers were not modeled in the analysis. The slope was modeled both without and with micropiles to evaluate the slope adjacent to and within the micropile stabilization area, respectively.

2. The Blackfoot River water surface elevation was assumed to have an average elevation of 3,240 feet. The groundwater surface elevation through the slope was assumed to be the same elevation as the river water surface.

3. Per the CH2MHill report, the micropile locations were assumed to be directly beneath the proposed trail and consisted of a pair of micropiles with a lateral spacing of 3 feet and pile shear strength of 56 kips for each pile.

4. A uniformly distributed construction traffic (and possibly future maintenance traffic) load across the trail was assumed to be approximately 250 psf.

5. Per the CH2MHill report, the subsurface profile consists of materials similar to those modeled in the report both in vertical and lateral extent across the slopes.

6. For the block analyses, the failure zone was assumed to occur within the clay sediment layer.

As discussed, the existing slope geometry, soil conditions, location of micropiles, and depths of soil layers were obtained from reports prepared by CH2MHill. Tetra Tech did not drill any soil borings or conduct additional laboratory testing for the slope analyses, as these tasks were not part of Tetra Tech's Scope of Work. For the analyses, Tetra Tech developed slope models of the existing slope using CH2MHill's soil strength and geometry data. Models were developed both with and without the inclusion of micropiles to demonstrate the impact of the trail system on the stability of the existing slopes outside the zone of micropile stabilization.

The following conclusions were obtained from the analyses;

1) The proposed trail system, including gabion walls and construction traffic and/or future maintenance vehicle loads will not significantly impact the stability of the existing slopes, either within or outside the zone of micropile stabilization, with a negligible reduction in the Factor of Safety of 0.02 for all models analyzed. We have addressed the requirement to minimize impacts to the slope during construction in the Excavations and Retaining Wall sections in this report.

2) The slope models developed by Tetra Tech for this study were based on parameters obtained from reports prepared by the Geotechnical Engineer of Record, CH2MHill. The slope models were evaluated solely to determine the impact the trail system would have on the stability of the existing slopes. The Geotechnical Engineer of Record should be consulted regarding any questions or concerns pertaining to the overall stability of the existing slope, irrespective of construction of the trail system.

The slope conditions modeled in the stability analyses are presented below in Table 2. Factors of safety for both circular and block failure analysis are presented for comparative purposes. Graphical plots of the critical stability analyses are presented in Figures 14 through 22 in the Appendix.

Table 2. Slope Stability Analyses Results

Analysis Condition	Average Water Elevation	
	Circular Failure Factor of Safety	Block Failure Factory of Safety
Existing slope ***without*** micropile reinforcement	1.02	*
Existing slope ***without*** micropile reinforcement, including proposed trail configuration	1.01	*
Existing slope ***without*** micropile reinforcement, including proposed trail configuration and construction and future maintenance traffic loads.	1.00	*
Existing slope ***with*** micropile reinforcement – Circular failure zone is on the river side of the micropile cap	1.02	1.34
Existing slope ***with*** micropile reinforcement, including proposed trail configuration	1.01	1.34
Existing slope ***with*** micropile reinforcement, including proposed trail configuration and construction and future maintenance traffic loads.	1.00	1.34

* Block Analysis was not performed as a block failure has not been previously identified within the unreinforced slope.

MRL Bridge West Slope

A slope stability analysis was also performed to evaluate the impact of the new trail on existing slope stability below the MRL bridge on the west side of the Blackfoot River. Slope geometries assumed in the stability analysis model for the slope below the MRL bridge were based on available design documents for the Milltown Trails, and Milltown Bridge Infrastructure Mitigation projects.

The existing slope topography and riprap geometry, were based on the cross-sections shown on plan sheet Nos. 1 and 2 from HDR's Pedestrian Path Feasibility report, dated March 26, 2009. The subsurface geometry and engineering properties of subsurface materials were based our field investigation and our previous project experience for the Highway 200 Bridge Design-Build and Bonner Pedestrian Bridge projects. Blackfoot River water surface elevations were based on Table 3-4 from CH2MHill's geotechnical report dated December 2006. The proposed trail configuration was based on Figures 5 and 6 of HDR's Preliminary Report dated September 30, 2011. Soil parameters used in the stability analyses are presented below in Table 3.

Table 3. Summary of Soil Parameters, Stability Analyses

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (degrees)
Embankment Fill	130	100	38
Gravel	130	0	36
Sandy Silt	110	100	30
Riprap	140	0	45
Gabion Fill	130	100	40

The stability analysis included the following assumptions:

1. The existing slope was modeled at a consistent slope angle of 1.5H:1V. The existing bridge piers were not modeled in the analysis.

2. Blackfoot River water surface elevation was assumed to have an average elevation of 3,241 feet. The groundwater surface elevation through the slope was assumed to be the same elevation as the river water surface.

3. The uniformly distributed construction traffic load across the trail was assumed to be approximately 250 psf.

4. The subsurface materials consist of materials similar to those modeled both in vertical and lateral extent across the slopes.

5. For design purposes, a factor of safety of 1.3 or greater is acceptable for static long-term stability.

The slope conditions modeled in the stability analyses are presented below in Table 4 with the results of the stability analyses. Graphic plots of the critical stability analyses are also presented in Figures 23 through 25 in the Appendix.

Table 4. Slope Stability Analyses Results

Analysis Condition	Factor of Safety
Existing slope	1.30
Existing slope including proposed trail configuration	1.29
Existing slope, including proposed trail configuration and traffic load	1.29

The analyses indicate that the existing slope is currently stable, and that the proposed trail configuration and construction traffic loads will not significantly impact slope stability, with a negligible decrease on the factor of safety.

CONTINUING SERVICES

Two additional elements of geotechnical engineering service are important to the successful completion of this project.

1. **Consultation with Tetra Tech during the design phase**. This is essential to ensure that the intent of our recommendations is incorporated in design decisions related to the project and that changes in the design concept consider geotechnical aspects.

2. **Observation and monitoring during construction**. Tetra Tech should be retained to observe the earthwork phases of the project, including the site grading and foundation excavations, to determine that the subsurface conditions are compatible with those used in our analysis and design. During site grading, placement of fill should be observed and tested to confirm that the proper compaction has been achieved. In addition, if environmental contaminants or other concerns are discovered in the subsurface, our personnel are available for consultation.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practices in the region where the work was conducted. The conclusions and recommendations submitted in this report are based upon project information provided to Tetra Tech, data obtained from the exploratory borings drilled at the locations indicated. The nature and extent of subsurface variations across the site may not become evident until construction. Tetra Tech should be on site during construction, to verify that actual subsurface conditions are consistent with those described herein.

This report has been prepared exclusively for our client. This report and the data included herein shall not be used by any third party without the express written consent of both the client and Tetra Tech. Tetra Tech is not responsible for technical interpretations by others. As the project evolves, we should provide continued consultation and field services during construction to review and monitor the implementation of our recommendations, and verify that our recommendations have been appropriately interpreted. Significant design changes may require additional analysis or modifications of the recommendations presented herein. We recommend on-site observation of excavations and foundation bearing strata and testing of fill by a representative of the geotechnical engineer.

Prepared by: Jeremy Dierking, P.E. Reviewed by: Marco Fellin, P.E.

APPENDIX

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the Geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A Geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting Geotechnical engineer indicates otherwise, *your Geotechnical engineer report should not be used:*

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified:
- when there is a change of ownership, or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their reports' development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by Geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no Geotechnical engineer, no matter how qualified, and not subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be fare more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their Geotechnical consultants through the construction stage*, to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a Geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a Geotechnical engineering report whose adequacy may have been affected by time.* Speak with the Geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as flood, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

GEOTECHNICAL SERVICES ARE PREFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems. *No individual other than the client should apply this report for its intended purpose without first conferring with the* *geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plants based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evalution of field samples. Only final boring logs customarily are included in geotechnical engineering reports. *These logs should not under any circumstances be redrawn* for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, *give contractors ready access to the complete geotechnical engineering report* prepared or authorized for their use. Those who do not provide such access may proceed under the *mistaken* impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are *not* exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE as developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

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TETRA TECH

LOGS OF EXPLORATIONS
EXPLANATION OF ABBREVIATIONS AND DESCRIPTIVE TERMS

SSS (SPT) - Standard penetration resistance test – results recorded as the number of blows of a 140-pound hammer falling 30 inches required to drive a 2-inch O.D. split sample spoon the second and third 6-inch increments of an 18-inch distance.

LSS - Modified penetration test – results recorded as the number of blows of a 140-pound hammer falling 30 inches required to drive a 2.5-inch O.D. split spoon the second and third 6-inch increments of an 18-inch distance.

SRS - Split barrel ring sampler 2-inches I.D. for taking undisturbed samples.

LRS - Split barrel ring sampler 2.5 inches I.D. for taking undisturbed samples.

STS - Shelby tube sampler for taking undisturbed samples (2" to 3-5/16" I.D.).
Sack (SK) or Bag - Sample of disturbed soil placed in canvas sack or plastic bag.

GWL - Groundwater level on the date shown on the logs.

RQD - Rock quality designation (RQD) for the bedrock samples are determined for each core run by summing the length of all sound, hard pieces of core over four inches in length, and dividing this number by the total length of the core run. This value, along with the core recovery percentage, is recorded on the drill logs.

GRAIN SIZES

	U.S. Standard Series Sieve					Clear Square Sieve Openings	
	200	40	10	4	¾"	3"	12"
Silts & Clays Distinguished on Basis of Plasticity	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

CONSISTENCY		RELATIVE DENSITY	
Clays & Silts	SPT* Blows/foot	Sands & Gravels	SPT* Blows/foot
Very Soft	0 – 2	Very Loose	0 – 4
Soft	3 – 4	Loose	5 – 10
Firm	5 – 8	Medium Dense	11 – 30
Stiff	9 – 15	Dense	31 – 50
Very Stiff	15 – 30	Very dense	Over 50
Hard	Over 30		

*Standard Penetration Test; PL = Plastic Limit; LL = Liquid Limit

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CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation: D 2487 – 83

(Based on Unified Soil Classification System)

MAJOR DIVISIONS				GROUP SYMBOL	GROUP NAME
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel [F]
		Gravels with Fines More than 12% fines	Fines classify as ML or MH	GM	Silty gravel [F G H]
			Fines classify as CL or CH	GC	Clayey gravel [F G H]
	Sands 50% or more of coarse faction passes No. 4 sieve	Clean Sands Less than 5% fines	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand [I]
		Sands with Fines More than 12% fines	Fines classify as ML or MH	SM	Silty Sand [G H I]
			Fines classify as CL or CH	SC	Clayey sand [G H I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line	CL	Lean clay [K L M]
			PI < 4 or plots below "A" line	ML	Silt [K L M]
		Organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OL	Organic clay [K L M N] Organic silt [K L M O]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay [K L M]
			PI plots below "A" line	MH	Elastic silt [K L M]
		Organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OH	Organic clay [K L M O] Organic silt [K L M O]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% require dual symbols:
- GW-GM well-graded gravel with silt
- GW-GC well-graded gravel with clay
- GP-GM poorly graded gravel with silt
- GP-GC poorly graded gravel with clay

[D] Sands with 5 to 12% fines require dual symbols:
- SW-SM well-graded sand with silt
- SW-SC well-graded sand with clay
- SP-SM poorly graded sand with silt
- SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $Cc=(D_{30})^2 / (D_{10} \times D_{90})$

[F] If soil contains ≥15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥15% gravel, add "with gravel" to group name.
If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.

[K]. If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

[L] If solid contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.

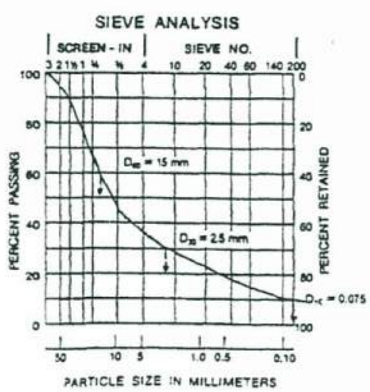
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

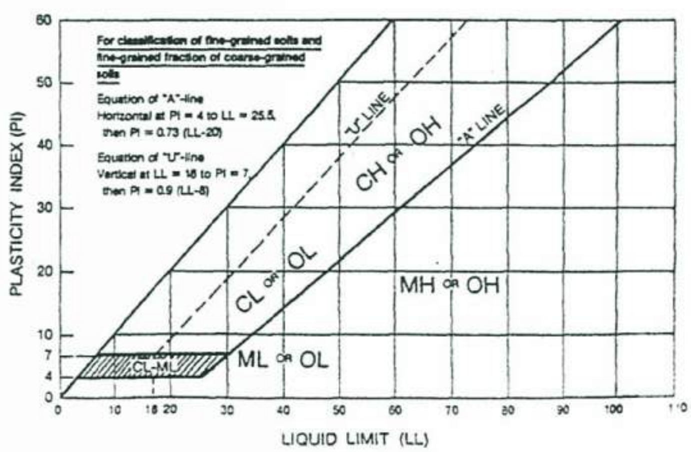
[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A: line.

[P] PI plots on or above "A: line.

[Q] PI plots below "A: line.





$C_u = \frac{D_{50}}{D_{1e}} = \frac{15}{0.075} = 200$ $C_c = \frac{(D_{30})2}{D_{12} \times 10_{36}} + \frac{(2.5)-}{0.075 \times 15} = 5.6$

G:\Environmental\Milltown\arcgis\location.mxd

Drawing No. 570451-1

Detailed Site Map

Confluence of the Clark Fork and Blackfoot Rivers

Milltown State Park

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Figure No. 1
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park		Project Number: 114-570451	
Borehole Location: Access Road		Borehole Number: BH-1	Sheet 1 of 1
Stationing:	Hammer: Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: Existing Grade		Notes:	

DEPTH (ft)	DRILL OPERATION	DRILL PRESSURE (psi)	DRILL RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
	Auger					Split Spoon	33	39-50/2"							Silty GRAVEL with sand, very dense, moist, brown, fine to medium grained gravel, subrounded gravel, fine to coarse grained sand, low plasticity.	4.00	
5	Auger					Split Spoon	33	22-50/5"	2						Silty GRAVEL with sand, very dense, moist, tan, fine to coarse grained gravel, subrounded to subangular gravel, fine grained sand, low plasticity.		
10	Auger					Split Spoon	25	11-22-22	2							10.50	

Bottom of boring at 10.5 feet

Operation Types: Auger; Mud Rotary; Air Rotary; Continuous Flight Auger; Diamond Core; Wash Rotary; Drive Casing

Sampler Types: Split Spoon; Penetrometer; Shelby; Vane Shear; Bulk Sample; California Ring; Grab Sample; Testpit

WATER LEVEL OBSERVATIONS

While Drilling ___ ft Upon Completion of Drilling ___ ft
Time After Drilling ___ ___ ___ ___
Depth To Water (ft) ___ ___ ___ ___
Remarks: Not Encountered

MILLTOWN STATE PARK.GPJ 2-28-12 CAM MONTANA DOT ENGLISH OUTPUT

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Figure No. 2
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park		Project Number: 114-570451	
Borehole Location: Access Road		Borehole Number: BH-2	Sheet 1 of 1
Stationing:	Hammer: Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: Existing Grade		Notes:	

DEPTH (ft)	DRILL OPERATION	DRILL PRESSURE (psi)	DRILL RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
	Auger					Drive Casing	33	24-50/3"							Silty SAND with gravel, very dense, moist, tan, fine to coarse grained gravel, subrounded to angular gravel, fine to coarse grained sands, low plasticity.		
						Bulk Sample					NV	NP	20				
						Drive Casing	66	14-21-16	1								
5																	
															Sandy SILT, stiff to hard, dry to moist, tan, fine grained sand, low plasticity.	7.00	
						Drive Casing	66	8-6-50/2"	2								
10																10.50	

Bottom of boring at 10.5 feet

Operation Types:		Sampler Types:	
Auger	Mud Rotary	Split Spoon	Penetrometer
Air Rotary	Continuous Flight Auger	Shelby	Vane Shear
Diamond Core	Wash Rotary	Bulk Sample	California Ring
Drive Casing		Grab Sample	Testpit

WATER LEVEL OBSERVATIONS

While Drilling ___ ft Upon Completion of Drilling ___ ft
Time After Drilling ___ ___ ___ ___
Depth To Water (ft) ___ ___ ___ ___
Remarks: Not Encountered

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Figure No. 3
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park			Project Number: 114-570451
Borehole Location: Pavillion		Borehole Number: BH-3	Sheet 1 of 1
Stationing:	Hammer: Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: Existing Grade		Notes:	

DEPTH (ft)	DRILL OPERATION	DRILL PRESSURE (psi)	DRILL RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
	Auger					Split Spoon	66	12-12-8							TOPSOIL.	0.30	
	Auger					Bulk Sample					20	4	18		Silty, clayey SAND with gravel, medium dense, moist, tan/brown, fine to medium grained gravel, subrounded gravel, fine to coarse grained sand, low plasticity.	3.00	
5	Auger					Split Spoon	33	22-12-9	5						Silty, clayey GRAVEL, medium dense to very dense, moist, brown, fine to medium grained gravel, subrounded gravel, fines have low to medium plasticity.		
10	Auger					Split Spoon	20	29-50/5"	4							10.00	
15	Auger					Split Spoon	10	50/3"	9						Silty GRAVEL with occasional boulders, very dense, moist, brown, fine to coarse grained gravel, rounded to subangular gravel, fines have low plasticity.	15.50	

Bottom of boring at 15.5 feet

Operation Types:		Sampler Types:	
Mud Rotary	Auger	Split Spoon	Penetrometer
Continuous Flight Auger	Air Rotary	Shelby	Vane Shear
Wash Rotary	Diamond Core	Bulk Sample	California Ring
	Drive Casing	Grab Sample	Testpit

WATER LEVEL OBSERVATIONS

While Drilling ___ ft Upon Completion of Drilling ___ ft
Time After Drilling ___ ___ ___ ___
Depth To Water (ft) ___ ___ ___ ___
Remarks: Not Encountered

MILLTOWN STATE PARK.GPJ ' 2-28-12 ' CAM ' MONTANA DOT ENGLISH OUTPUT

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Figure No. 4
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park		Project Number: 114-570451	
Borehole Location: Parking Area		Borehole Number: BH-4	Sheet 1 of 1
Stationing:	Hammer Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: Existing Grade		Notes:	

DEPTH (ft)	DRILL: OPERATION	DRILL: PRESSURE (psi)	DRILL: RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
0	Auger					Split Spoon	30	50/5"							Poorly graded GRAVEL, very dense, wet, gray, medium to coarse grained gravel, angular gravel.	1.00	
	Auger														Silty GRAVEL, very dense, moist, tan/brown, fine to medium grained gravel, subrounded gravel, low plasticity.	3.00	
5	Auger					California Ring	90	21-27-30							Poorly graded GRAVEL with silt and sand and occasional boulders, very dense, fine to coarse grained gravel, subrounded to subangular, fine to coarse grained sand, low plasticity.	7.00	
10	Auger					California Ring	90	4-5-5							Sandy SILT, stiff, moist, gray, fine to medium grained sand, no to low plasticity.	10.50	

Bottom of boring at 10.5 feet

Operation Types: Mud Rotary; Continuous Flight Auger; Wash Rotary; Auger; Air Rotary; Diamond Core; Drive Casing

Sampler Types: Split Spoon; Shelby; Bulk Sample; Grab Sample; Penetrometer; Vane Shear; California Ring; Testpit

WATER LEVEL OBSERVATIONS

While Drilling ____ ft Upon Completion of Drilling ____ ft
Time After Drilling
Depth To Water (ft)
Remarks: Not Encountered

MILLTOWN STATE PARK.GPJ · 2-28-12 · CAM · MONTANA DOT ENGLISH OUTPUT

Revised 5-17-11 (MAT)

Tetra Tech
2525 Palmer Street, Suite 2
Missoula, MT 59808
Phone: (406) 543-3045
Fax: (406) 543-3088

Figure No. 5
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park			Project Number: 114-570451
Borehole Location: MRL Bridge		Borehole Number: BH-5	Sheet 1 of 2
Stationing:	Hammer: Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: 3265.00		Notes:	

DEPTH (ft)	DRILL OPERATION	DRILL PRESSURE (psi)	DRILL RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
	Auger														TOPSOIL.	0.30	
	Auger					Split Spoon	33	5-11-29	2						Silty GRAVEL with sand, medium dense to dense, moist, brown, fine to coarse grained gravel, subangular gravel, fine grained sand, no to low plasticity.		
5	Auger					Split Spoon	33	12-8-22	2							7.00	
	Auger														Sandy SILT, stiff, slightly moist, gray, fine to coarse grained sand, no to low plasticity.		
10	Auger					Split Spoon	66	4-5-5	5								
15	Auger					California Ring	33	9-50/3"								15.50	
	Auger														Poorly graded GRAVEL with sand, dense, moist, gray, fine to coarse grained gravel, subangulargravel, fine to medium grained sand.	17.50	
	Auger														Sandy SILT, stiff, moist, gray, fine to medium grained sand, no to low plasticity.		
20	Auger					California Ring	66	4-4-5								22.00	

Operation Types: Mud Rotary; Continuous Flight Auger; Wash Rotary; Auger; Air Rotary; Diamond Core; Drive Casing

Sampler Types: Split Spoon; Shelby; Bulk Sample; Grab Sample; Penetrometer; Vane Shear; California Ring; Testpit

WATER LEVEL OBSERVATIONS

While Drilling 28.00 ft Upon Completion of Drilling ft
Time After Drilling
Depth To Water (ft)
Remarks: Not Encountered

MILLTOWN STATE PARK.GPJ ' 2-28-12 ' CAM ' MONTANA DOT ENGLISH OUTPUT

Revised 5-17-11 (MAT)

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Missoula, MT 59808
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Fax: (406) 543-3088

Figure No. 5
LOG OF BORING

TETRA TECH

Project Name: Milltown State Park		Project Number: 114-570451	
Borehole Location: MRL Bridge		Borehole Number: BH-5	Sheet 2 of 2
Stationing:	Hammer Type: Automatic	Driller: O'Keefe - Butte	Logger: Kyle Zanto
Drilling Equipment: Mobile B-61	Borehole Diameter (in): 8.00	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: 3265.00		Notes:	

DEPTH (ft)	DRILL: OPERATION	DRILL: PRESSURE (psi)	DRILL: RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
	Auger														Poorly graded GRAVEL with silt and sand, very dense, damp, gray, fine to coarse grained gravel, subangular gravel, fine to coarse grained sand, no to low plasticity, occasional boulders or cobbles.		
25						Split Spoon	33	25-50/3"	3								
30						Split Spoon	50	19-29-17	10							28.00 (water, while drilling)	
35						Split Spoon	40	7-8-14	9								
40						Split Spoon	40	9-50/5"	1							40.50	

Bottom of boring at 40.5 feet

Operation Types: Mud Rotary; Continuous Flight Auger; Wash Rotary; Auger; Air Rotary; Diamond Core; Drive Casing

Sampler Types: Split Spoon; Shelby; Bulk Sample; Grab Sample; Penetrometer; Vane Shear; California Ring; Testpit

WATER LEVEL OBSERVATIONS

While Drilling 28.00 ft Upon Completion of Drilling ___ ft
Time After Drilling
Depth To Water (ft)
Remarks: Not Encountered

MILLTOWN STATE PARK.GPJ · 2-28-12 · CAM · MONTANA DOT ENGLISH OUTPUT

Revised 5-17-11 (MAT)

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Fax: (406) 543-3088

Figure No. 6
LOG OF TEST PIT

TETRA TECH

Project Name: Milltown State Park		Project Number: 114-570451	
Test Pit Location: MRL Bridge	Test Pit Number: TP-1	Sheet 1 of 1	
Stationing:	Operator: High Country Excavating	Logger: Kyle Zanto	
Excavation Type: CAT 310	Pit Dimensions: '(w) x '(l) x '(d)	Date Started: 1/10/12	Date Finished: 1/10/12
Elevation and Datum: Ground: 3274.00	Notes:		

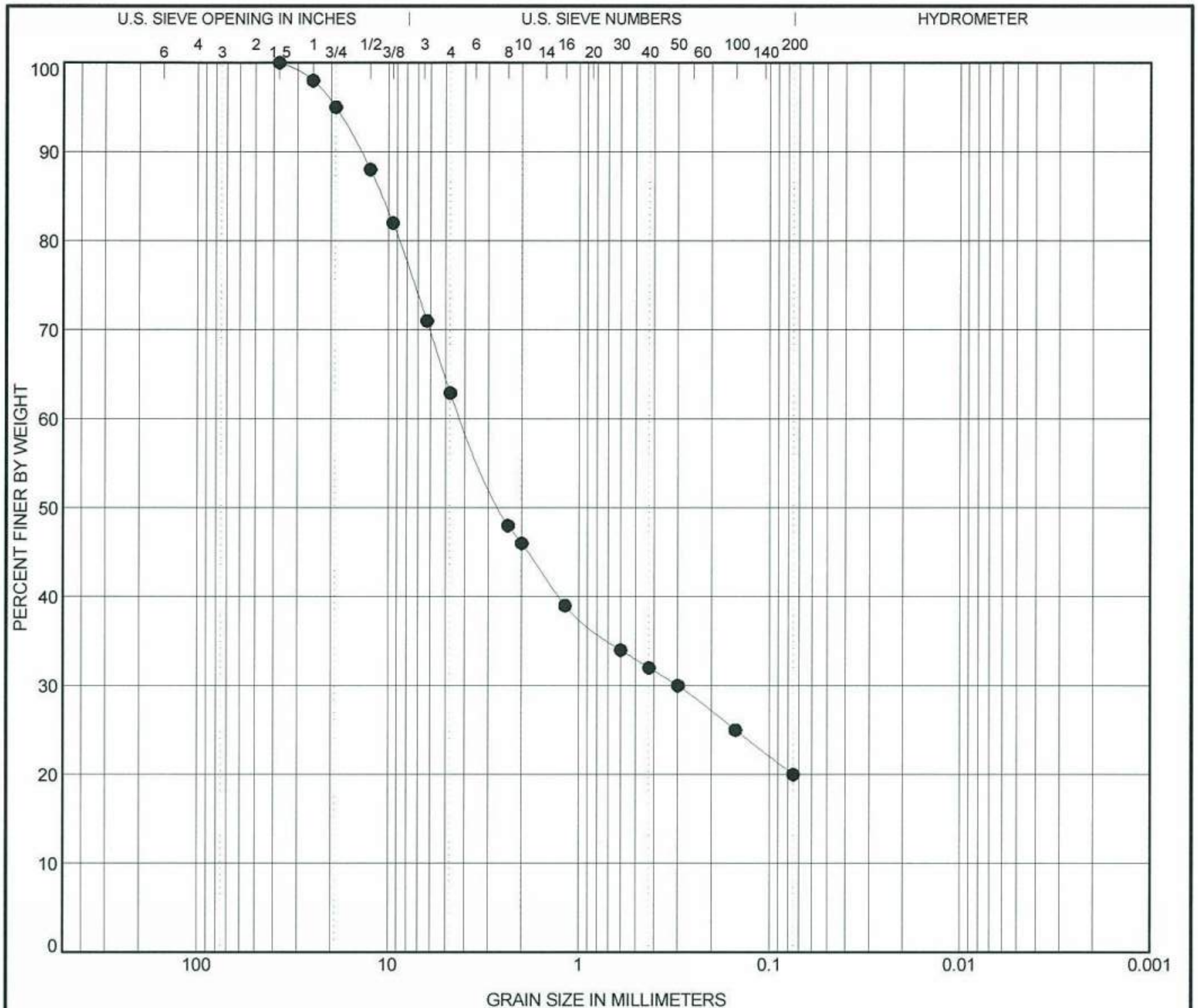
DEPTH (ft)	DRILL: OPERATION	DRILL: PRESSURE (psi)	DRILL: RATE (mph)	CORE PERCENT RECOVERY	ROCK QUALITY DESIGNATION (RQD)	SAMPLE	RECOVERY (%)	STANDARD PENETRATION TEST SPT	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LIQUID LIMIT LL	PLASTICITY INDEX PI	MINUS NO. 200 (%)	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH (ft)	REMARKS
0–8						Bulk Sample					NV	NP	20		Fill, silty GRAVEL with sand, cobbles, and boulders up to 18", medium dense, damp, brown, fine to coarse grained gravel, subangular to angular gravel, fine to coarse grained sand, no to low plasticity.	8.00	
8–10						Bulk Sample									Silty GRAVEL with sand, medium dense to dense, dark brown, fine to coarse grained gravel, subrounded to subangular gravel, fines have no to low plasticity.	10.00	

Bottom of test pit at 10 feet

Operation Types:		Sampler Types:	
Mud Rotary	Auger	Split Spoon	Penetrometer
Continuous Flight Auger	Air Rotary	Shelby	Vane Shear
Wash Rotary	Diamond Core	Bulk Sample	California Ring
	Excavation	Grab Sample	Testpit

WATER LEVEL OBSERVATIONS

While Drilling ___ ft   Upon Completion of Drilling ___ ft
Time After Drilling
Depth To Water (ft)
Remarks: Not Encountered



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	LL	PL	PI	Cc	Cu
BH-2 - (1 - 5 ft)	**SILTY SAND with GRAVEL(SM)**	**NV**	**NV**	**NP**		

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
BH-2 - (1 - 5 ft)	**37.5**	**4.145**	**0.3**		**37**	**43**	**20**	

MILLTOWN STATE PARK.GPJ 2-28-12 CAM TT_US GRAIN SIZE

TETRA TECH

GRAIN SIZE DISTRIBUTION

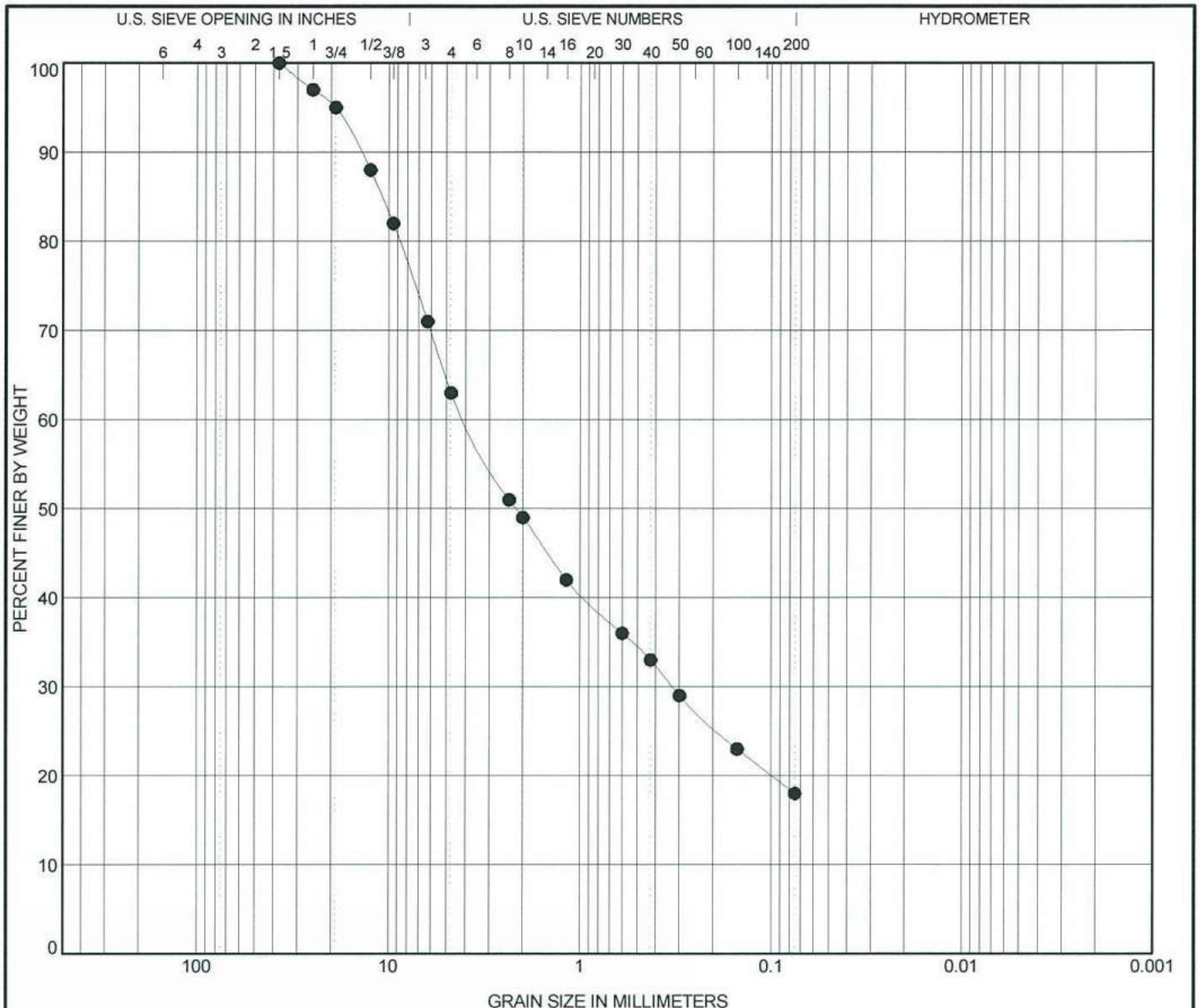
Project: Milltown State Park

Location: Access Road

Number: 114-570451

Figure No. 7

Revised 1-23-08 (MAT)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	LL	PL	PI	Cc	Cu
BH-3 - (1 - 3 ft)	**SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	**20**	**16**	**4**		

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
BH-3 - (1 - 3 ft)	**37.5**	**3.988**	**0.327**		**37**	**45**	**18**	

MILLTOWN STATE PARK.GPJ 2-28-12 CAM TT_US GRAIN SIZE

TETRA TECH

GRAIN SIZE DISTRIBUTION

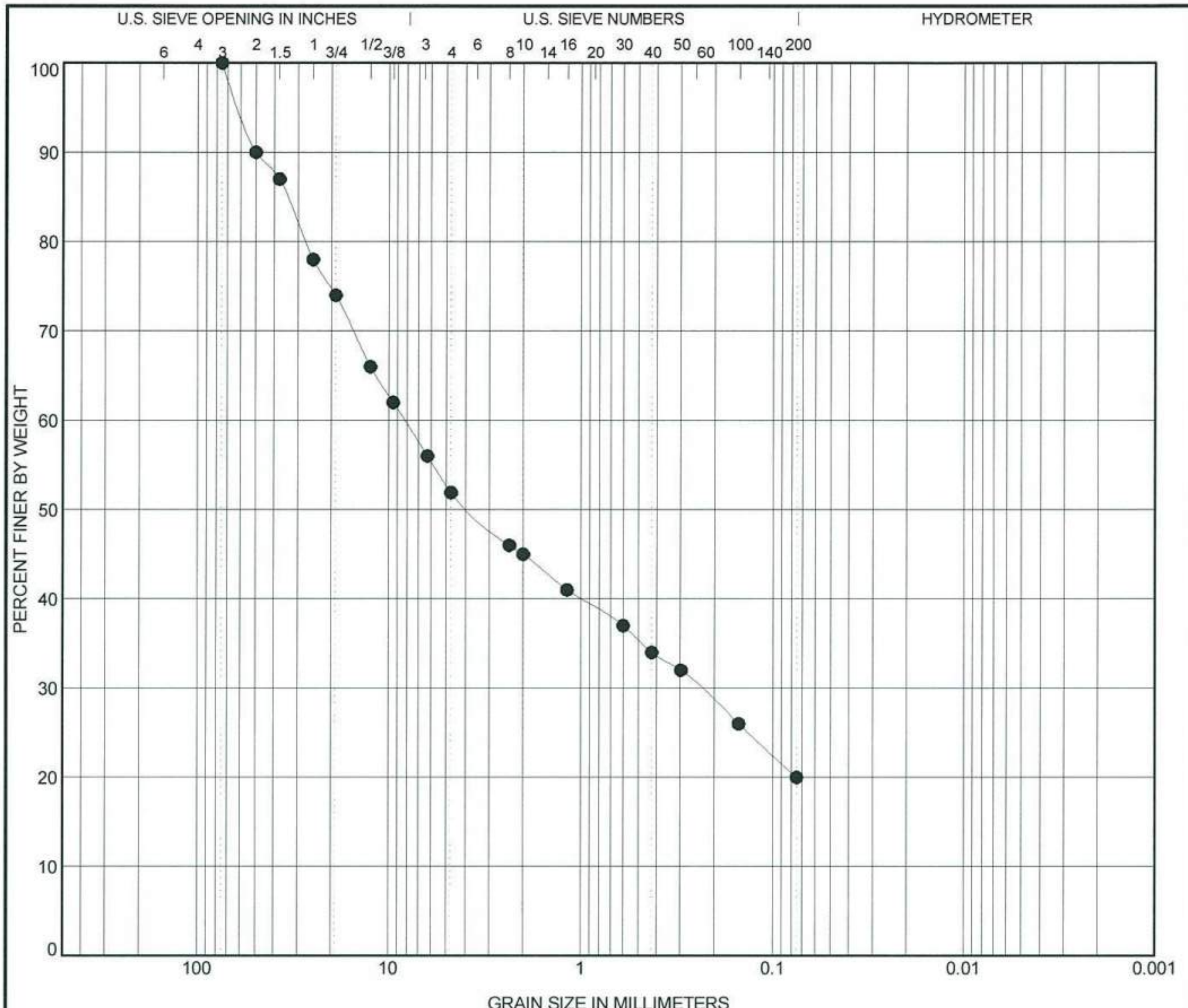
Project: Milltown State Park

Location: Pavillion

Number: 114-570451

Figure No. 8

Revised 1-23-08 (MAT)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	LL	PL	PI	Cc	Cu
TP-1 - (5 - 7 ft)	**SILTY GRAVEL with SAND(GM)**	**NV**	**NV**	**NP**		

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
TP-1 - (5 - 7 ft)	**75**	**8.284**	**0.238**		**48**	**32**	**20**	

MILLTOWN STATE PARK.GPJ ' 2-28-12 ' CAM ' TT_US GRAIN SIZE

TETRA TECH

GRAIN SIZE DISTRIBUTION

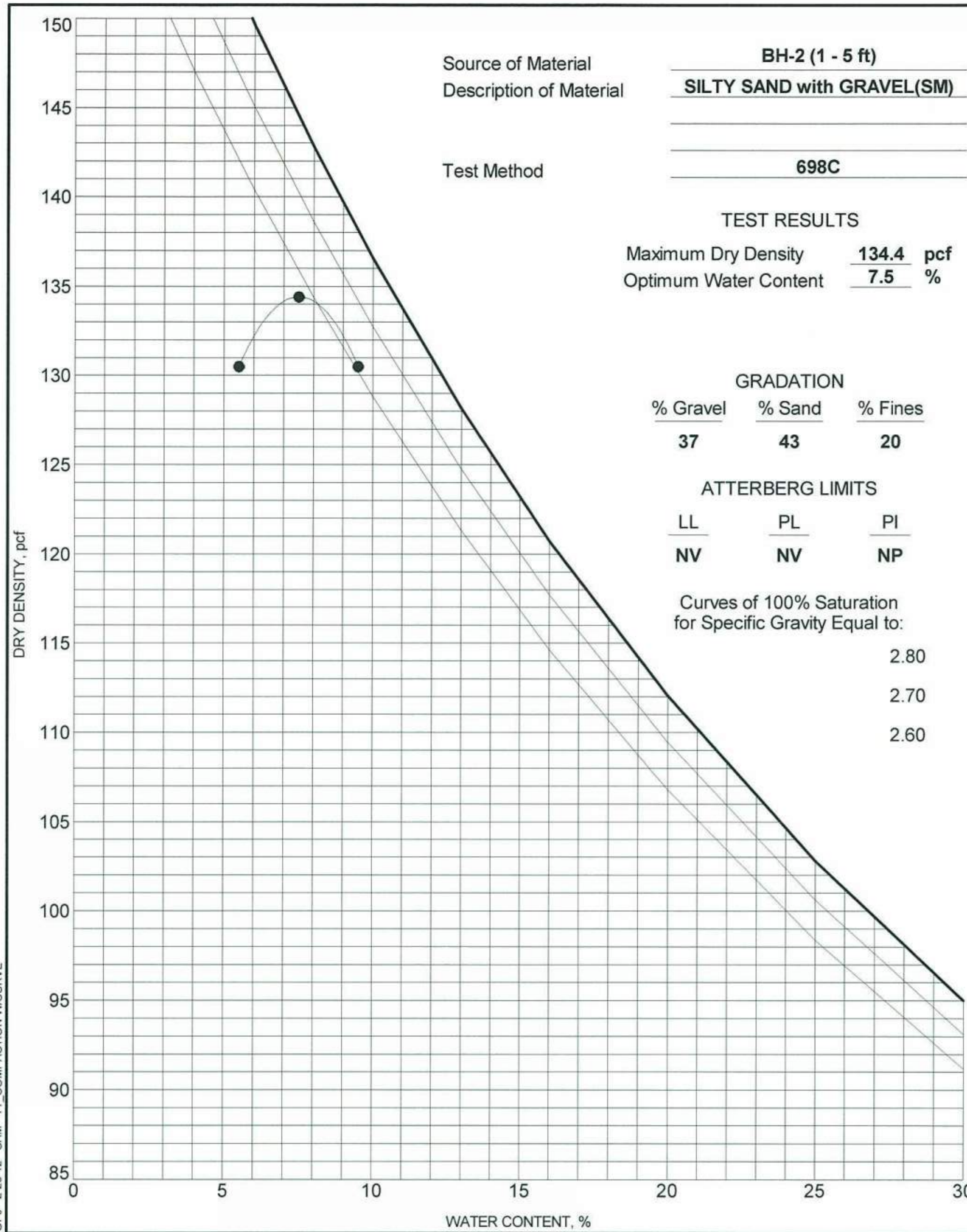
Project: Milltown State Park

Location: MRL Bridge

Number: 114-570451

Figure No. 9

Revised 1-23-08 (MAT)



Source of Material: BH-2 (1 - 5 ft)

Description of Material: SILTY SAND with GRAVEL(SM)

Test Method: 698C

TEST RESULTS

Maximum Dry Density 134.4 pcf

Optimum Water Content 7.5 %

GRADATION

% Gravel	% Sand	% Fines
37	43	20

ATTERBERG LIMITS

LL	PL	PI
NV	NV	NP

Curves of 100% Saturation for Specific Gravity Equal to:

2.80

2.70

2.60

MILLTOWN STATE PARK.GPJ ' 2-28-12 ' CAM ' TT_COMPACTION W/CURVE

MOISTURE-DENSITY RELATIONSHIP

TETRA TECH

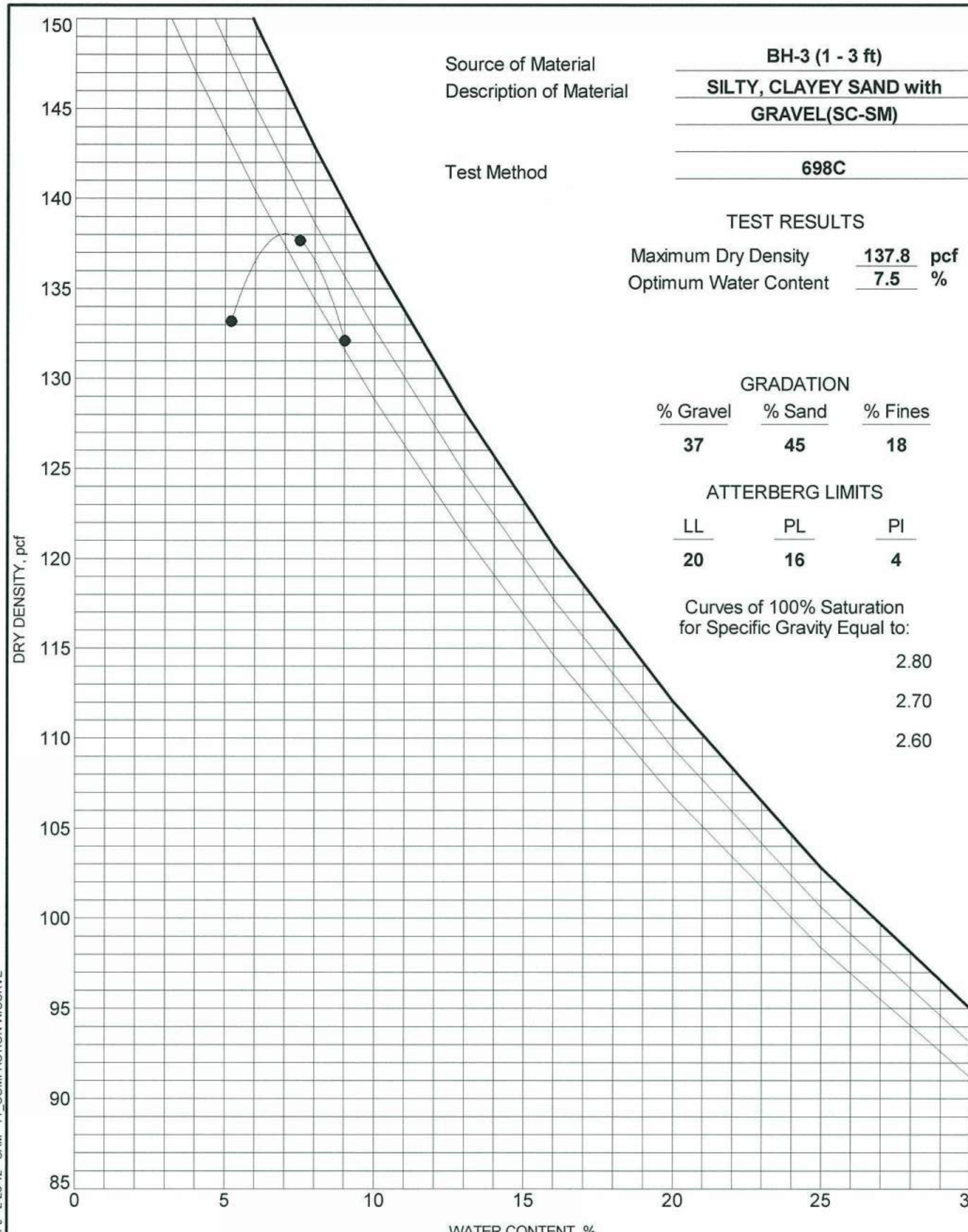
Project: Milltown State Park

Location: Access Road

Number: 114-570451

Figure No. 10

Revised 1-23-08 (MAT)



Source of Material	BH-3 (1 - 3 ft)
Description of Material	SILTY, CLAYEY SAND with GRAVEL(SC-SM)
Test Method	698C

TEST RESULTS

Maximum Dry Density	137.8	pcf
Optimum Water Content	7.5	%

GRADATION

% Gravel	% Sand	% Fines
37	45	18

ATTERBERG LIMITS

LL	PL	PI
20	16	4

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

MILLTOWN STATE PARK.GPJ ' 2-28-12 ' CAM ' TT_COMPACTION W/CURVE

MOISTURE-DENSITY RELATIONSHIP

TETRA TECH

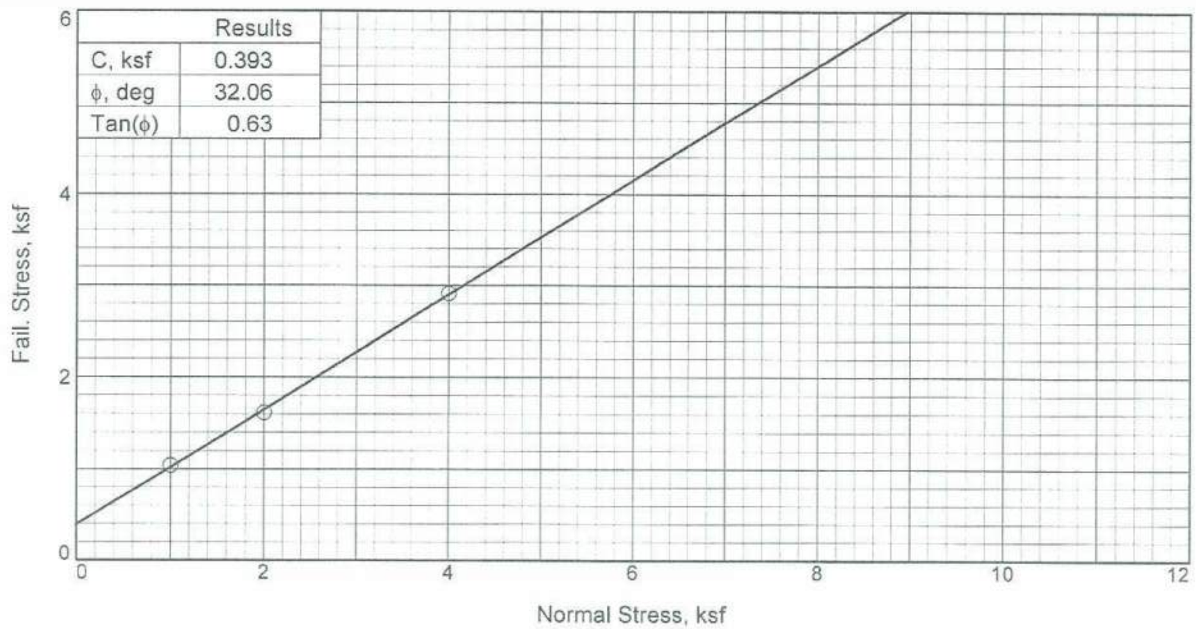
Project: Milltown State Park

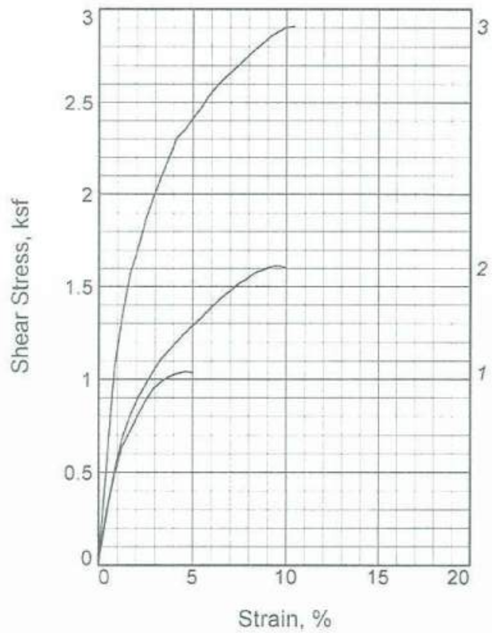
Location: Pavillion

Number: 114-570451

Figure No. 11

Revised 1-23-08 (MAT)





Sample No.		1	2	3
Initial	Water Content, %	12.2	12.2	12.2
	Dry Density, pcf	102.3	103.2	101.9
	Saturation, %	52.3	53.6	51.9
	Void Ratio	0.6175	0.6030	0.6231
	Diameter, in.	2.40	2.40	2.40
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.0	19.9	18.2
	Dry Density, pcf	104.3	106.2	110.0
	Saturation, %	90.5	94.6	95.7
	Void Ratio	0.5858	0.5581	0.5038
	Diameter, in.	2.40	2.40	2.40
	Height, in.	0.98	0.97	0.93
Normal Stress, ksf		1.000	2.000	4.000
Fail. Stress, ksf		1.041	1.614	2.909
Strain, %		4.6	9.6	10.4
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.01	0.01	0.01

Sample Type: Undisturbed
Description: Silty SAND

LL= NV **PI=** NP
Assumed Specific Gravity= 2.65
Remarks:

Client:

Project: MILLTOWN STATE PARK

Source of Sample: BH-5 **Depth:** 19.0'-20.5'

Proj. No.: 114-570451 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Tetra Tech, Inc.
Billings, MT

Figure 12

TETRA TECH

PROJECT:	MILLTOWN STATE PARK	**FIGURE**	13
LOCATION:	MISSOULA , MONTANA	**PROJECT NO:**	114-570451
MATERIAL:	SILTY SAND WITH GRAVEL	**WORK ORDER NO:**	1
SAMPLE SOURCE:	BH-2 FROM 1 TO 5'	**LAB NO:**	3
REVIEWED BY:	M.CONNOLLY	**DATE SAMPLED:**	1/12/2012

CBR(CALIFORNIA BEARING RATIO) OF LABORATORY-COMPACTED SOILS(ASTM D1883)

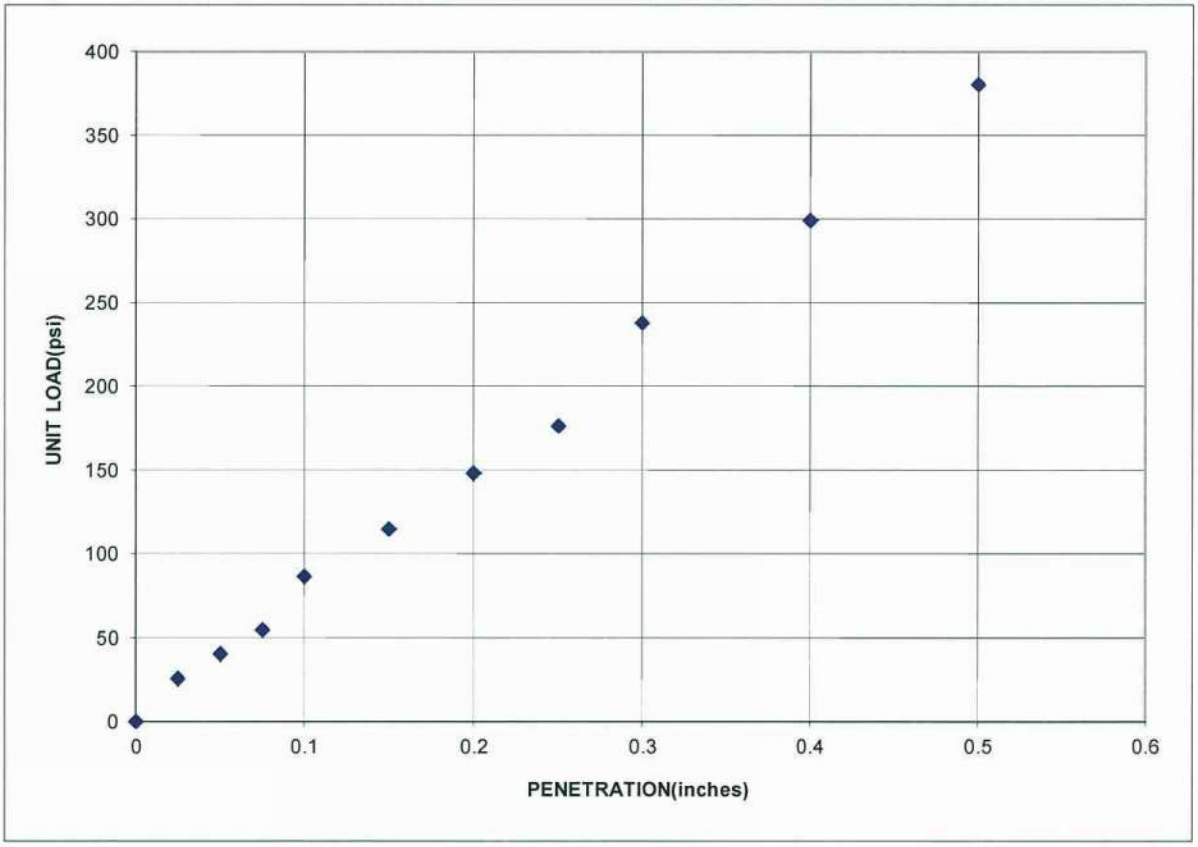
COMPACTION(%) 95.0
COMPACTION: 3 LIFTS @ BLOWS/LIFT
PERCENT SWELL -0.04%

PENETRATION	CORRECTED C B R
0.100	9
0.200	10

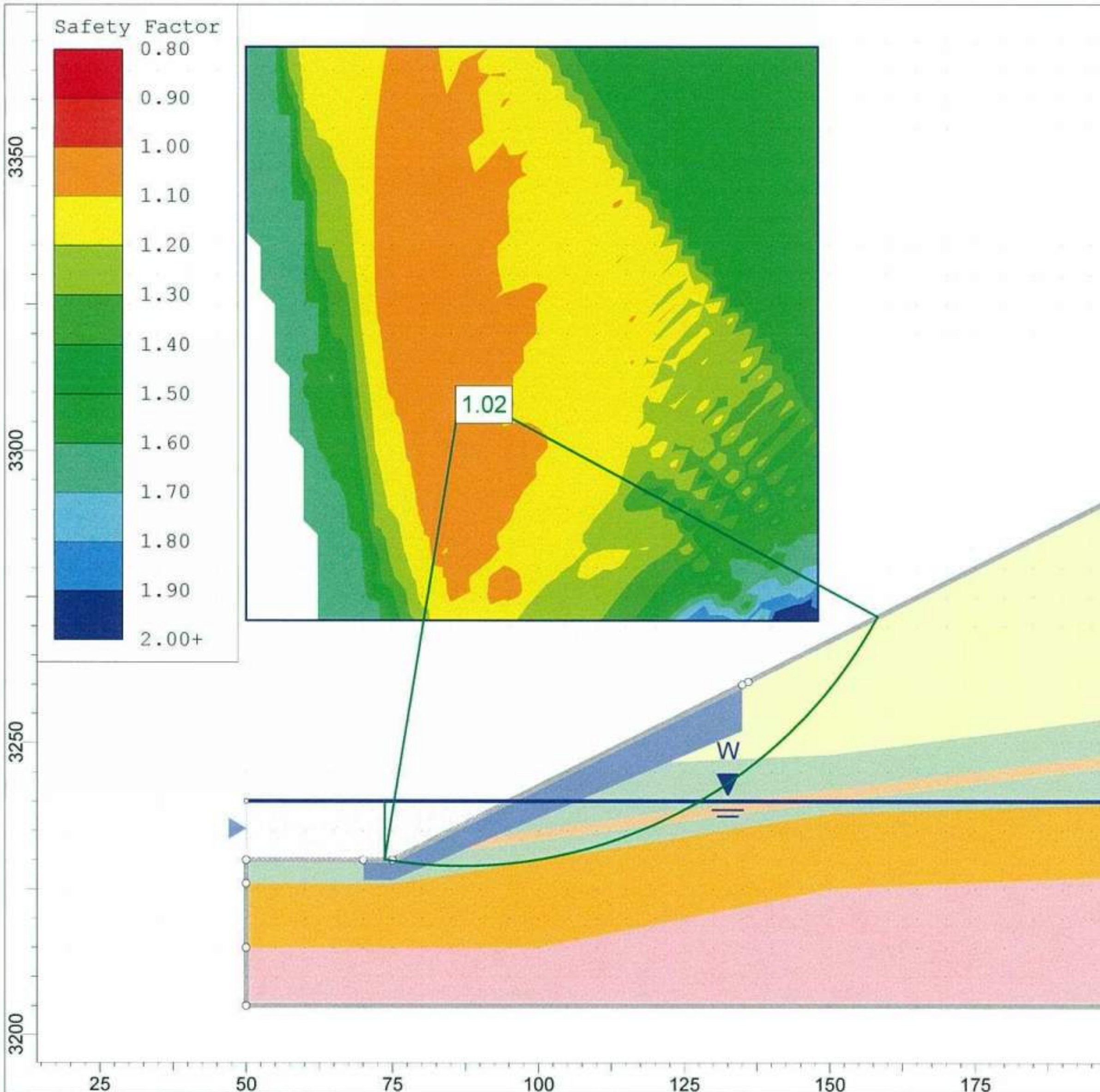
	BEFORE SOAK	AFTER SOAK
DRY DENSITY	127.7 lbs./cu.ft	127.0 lbs./cu.ft
PERCENT MOISTURE	9.5 %	10.7 %

D698C PROCTOR
DRY DENSITY(pcf) 134.4
MOISTURE(%) 7.5

SURCHARGE WEIGHT 10 lbs.



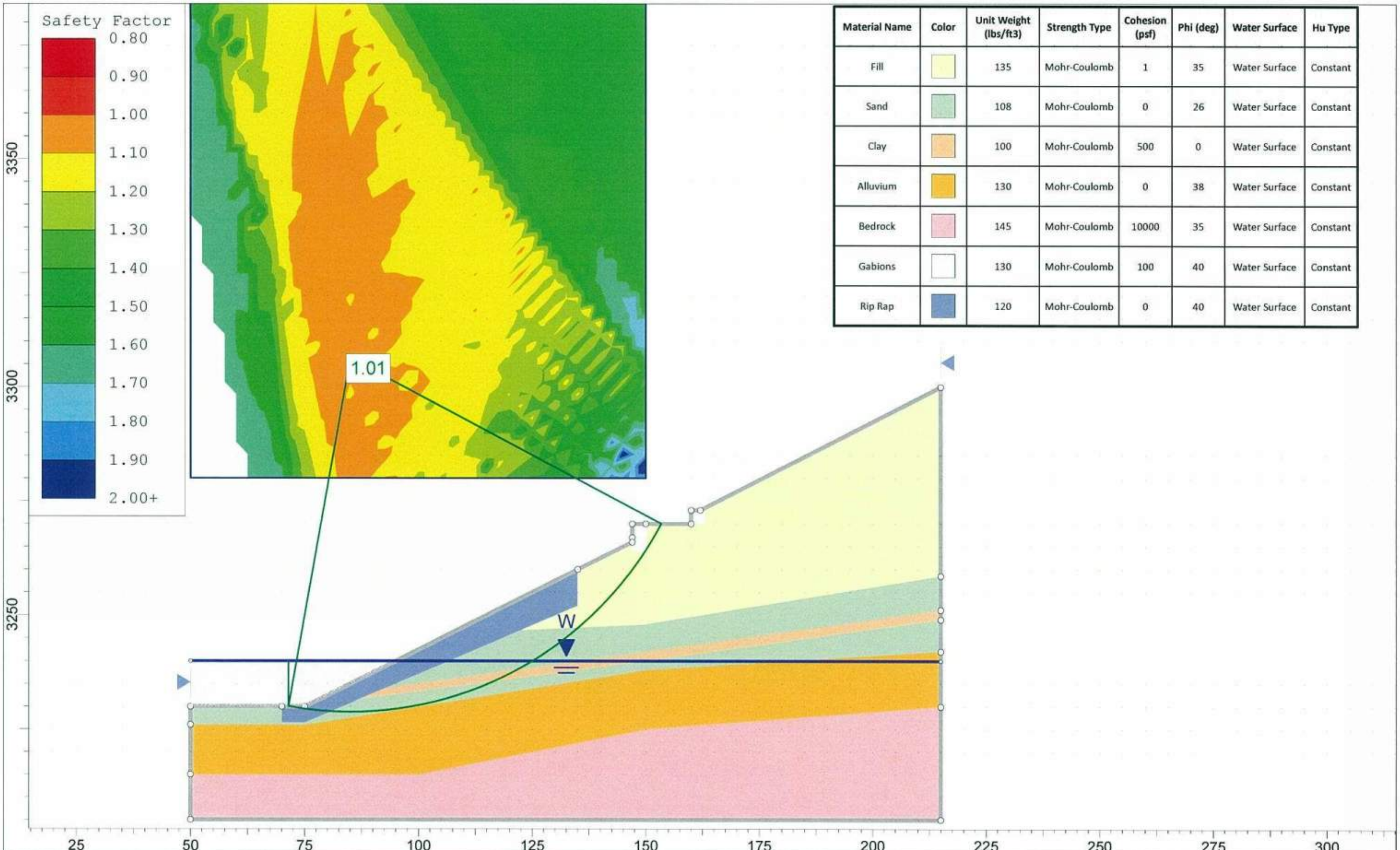
CBR 9



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

rocscience	Project	Milltown Trails, I-90 Bridges West Slope	
	Analysis Description	Existing 2H:1V Slope, Average Water Elev, Circular Analysis	
	Drawn By: JRD	Scale: 1:350	Company: Tt
SLIDEINTERPRET 6.015	Date: 2/20/2012		File Name: I-90 slope-1c.slim

Figure 14

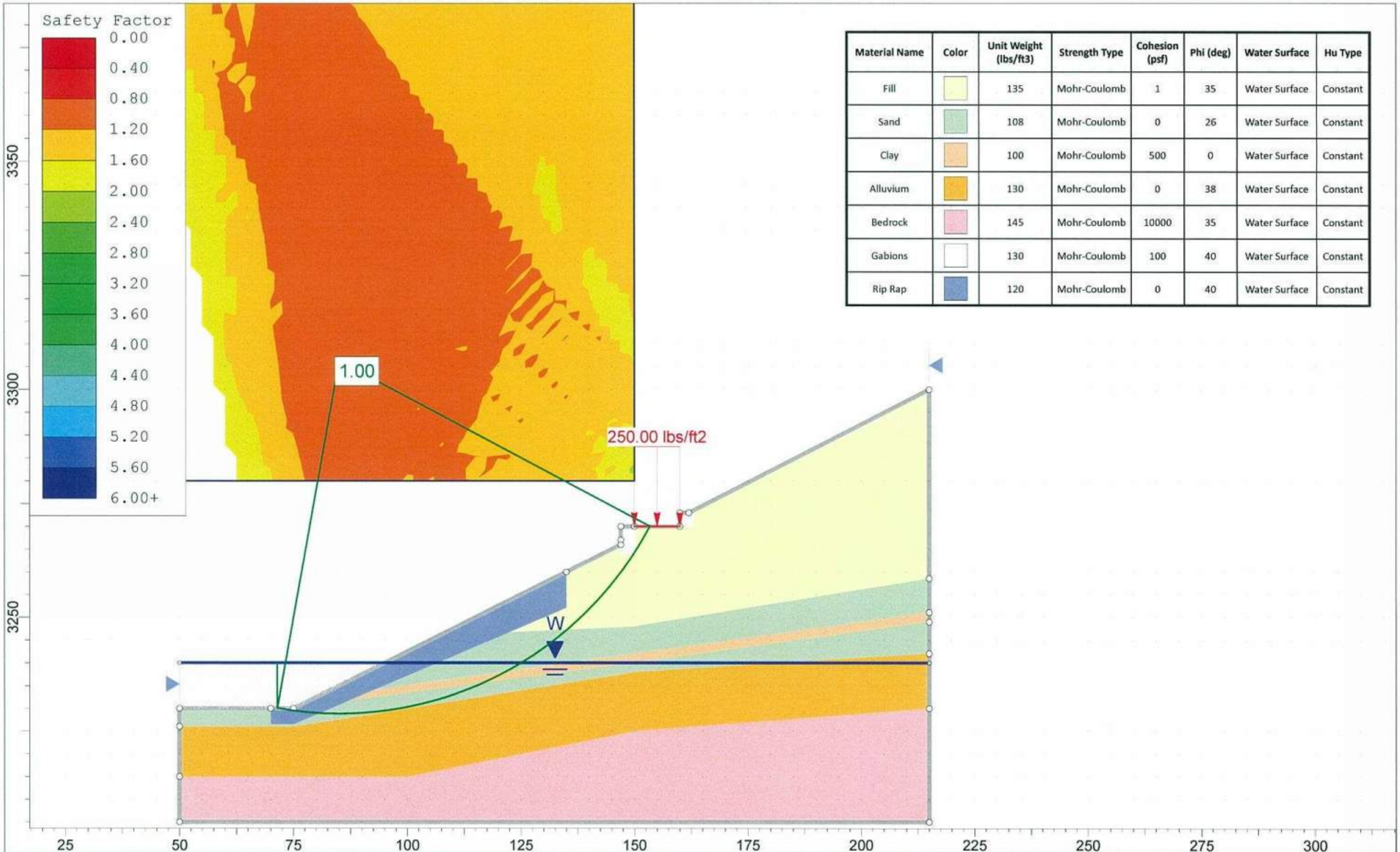


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

SLIDEINTERPRET 6.015

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope, Average Water Elev, with Trail, Circular Analysis		
Drawn By JRD	*Scale* 1:350	*Company*	Tt
Date	2/20/2012	*File Name*	I-90 slope-2c.slim

Figure 15



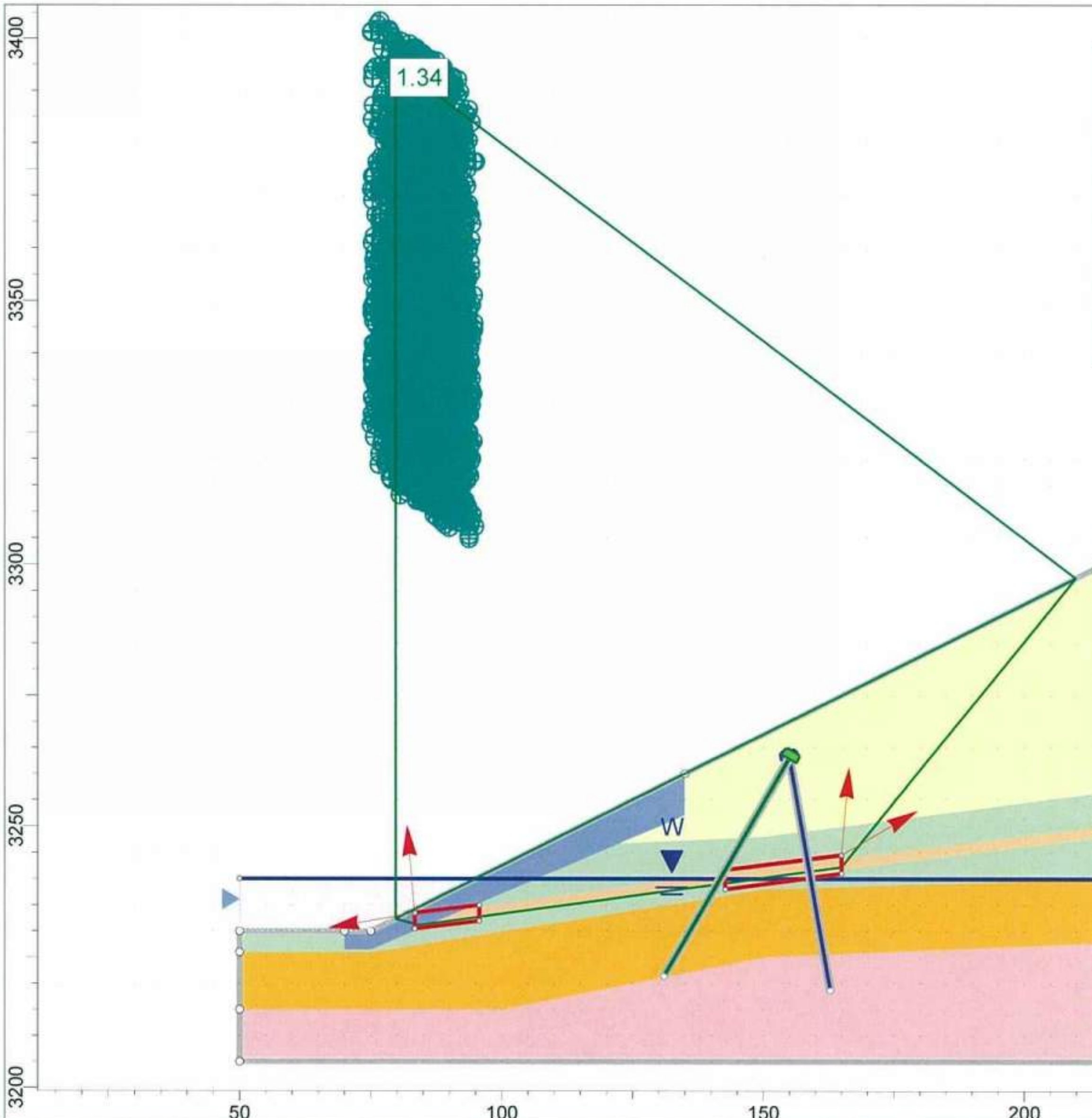
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

rocscience

SLIDEINTERPRET 6.015

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope, Average Water Elev, with Trail and Traffic Load, Circular Analysis		
Drawn By	JRD	Scale	1:350
Company	Tt		
Date	2/20/2012	File Name	I-90 slope-2ct.slim

Figure 16



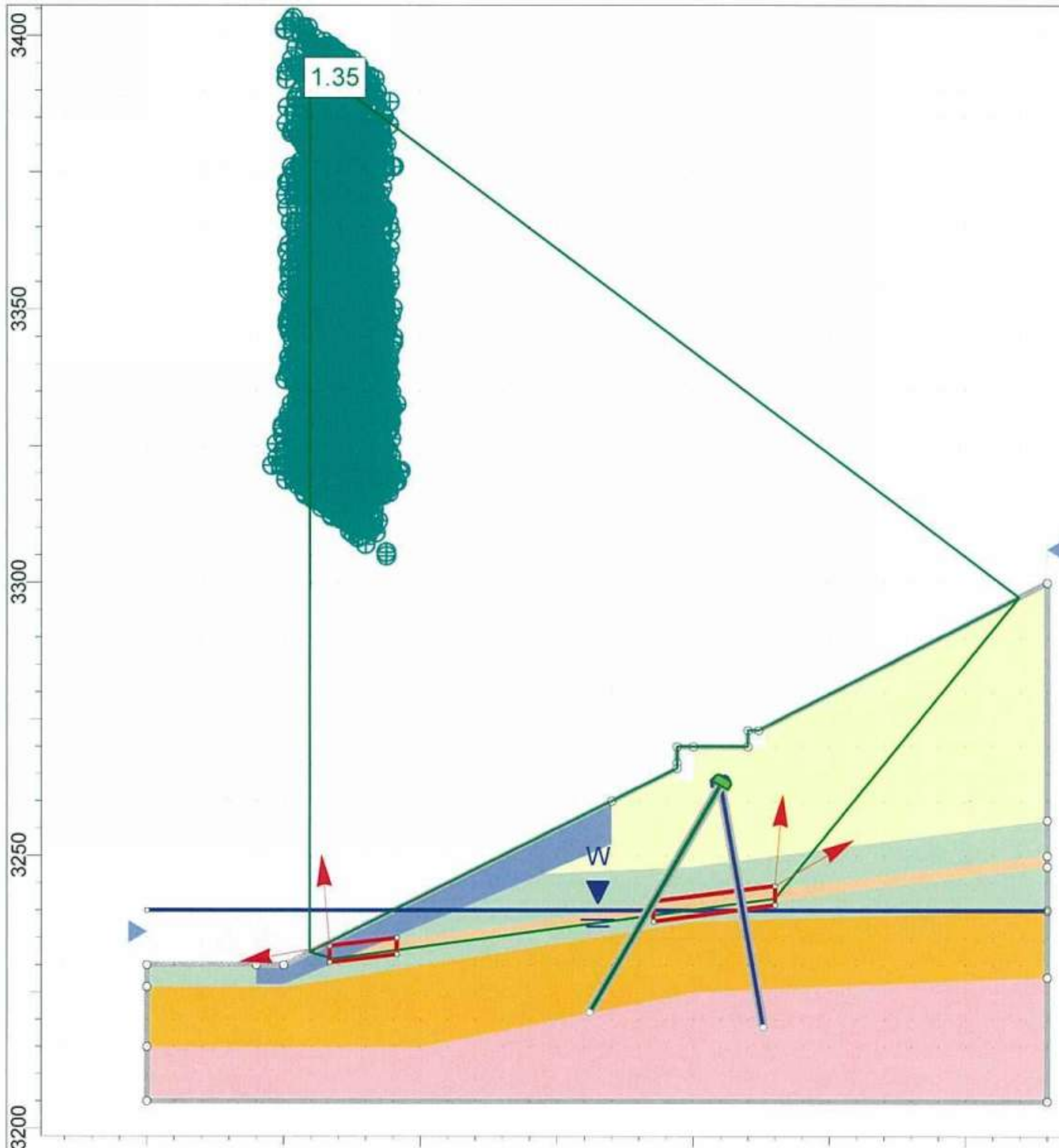
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, Block Analysis		
Drawn By	JRD	Scale	1:400
Company	Tt		
Date	2/20/2012	File Name	I-90 slope-1m.slim

SLIDEINTERPRET 6.015

Figure 17



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

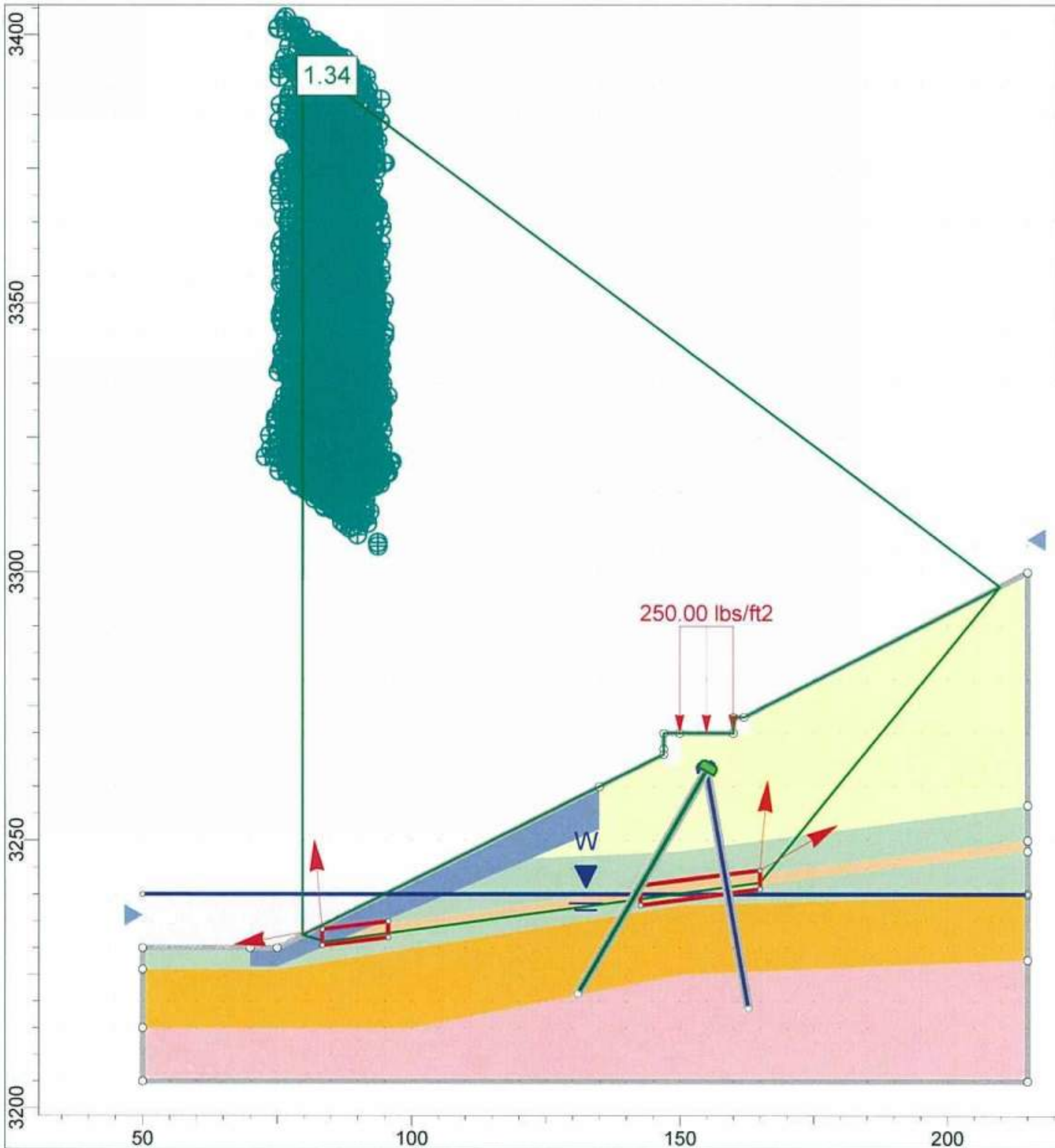
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

rocscience

SLIDEINTERPRET 6.015

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, with Trail, Block Analysis		
Drawn By JRD	*Scale* 1:400	*Company*	Tt
Date	2/20/2012	*File Name*	I-90 slope-2mb.slim

Figure 18

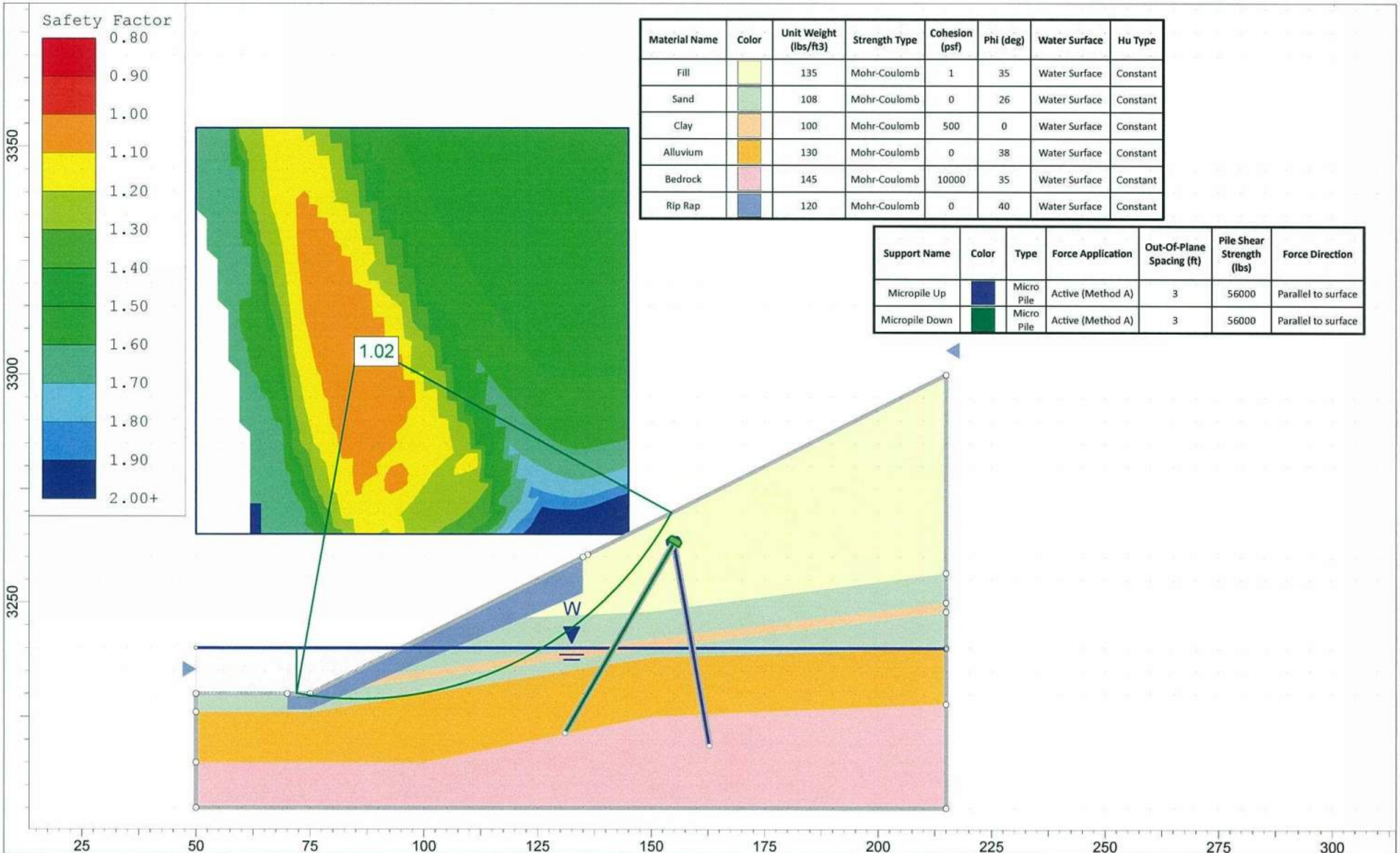


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

rocscience SLIDEINTERPRET 6.015	*Project*	Milltown Trails, I-90 Bridges West Slope		
	Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, with Trail and Traffic Load, Block Analysis		
	Drawn By JRD	*Scale* 1:400	*Company*	Tt
	Date 2/20/2012		*File Name*	I-90 slope-2mb.slim

Figure 19



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

Project	Milltown Trails, I-90 Bridges West Slope
Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, Circular Analysis
Drawn By JRD	*Scale* 1:350
Company	Tt
Date 2/20/2012	*File Name* I-90 slope-1cm.slim

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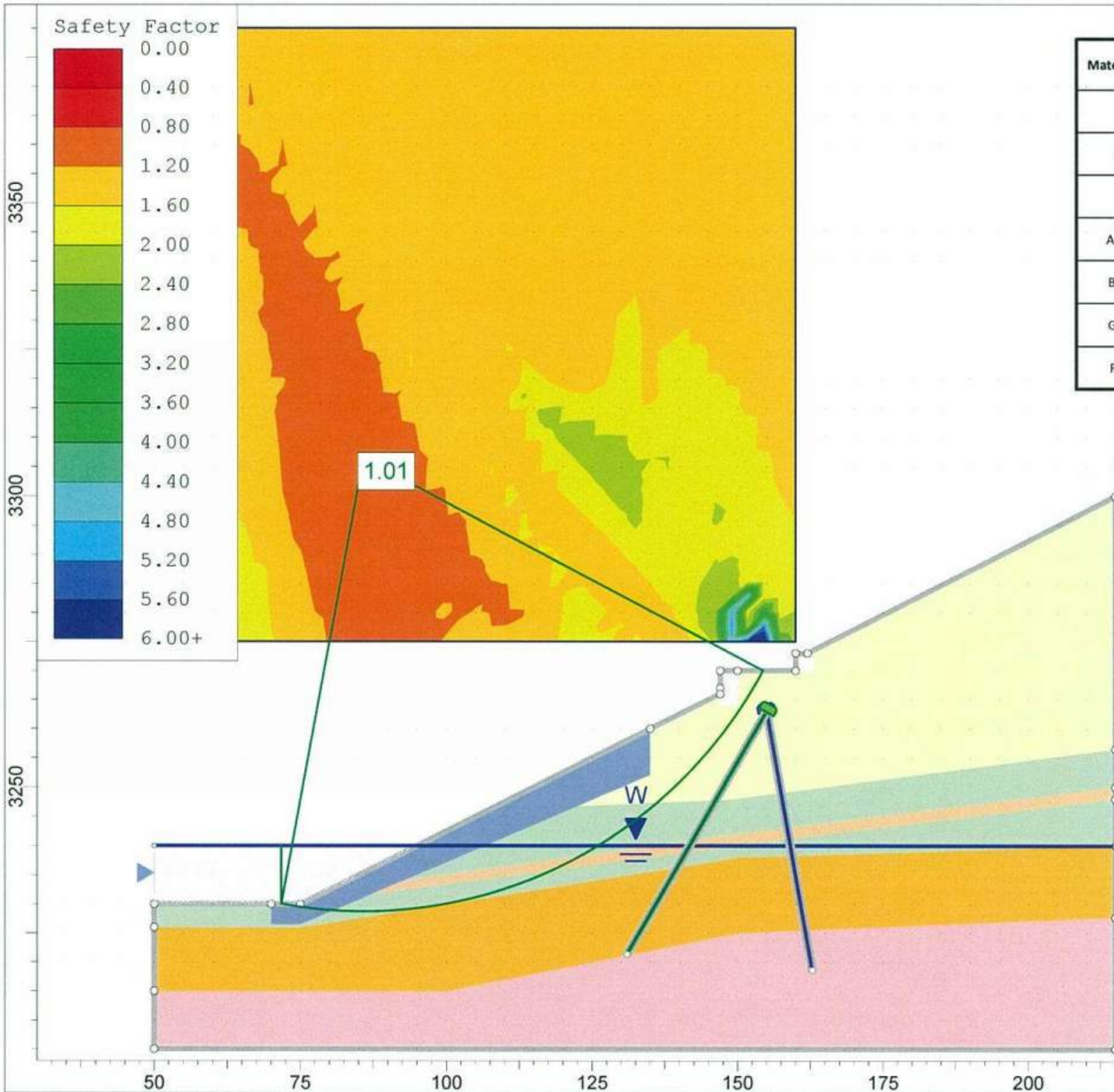
SLIDEINTERPRET 6.015

Figure 20



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

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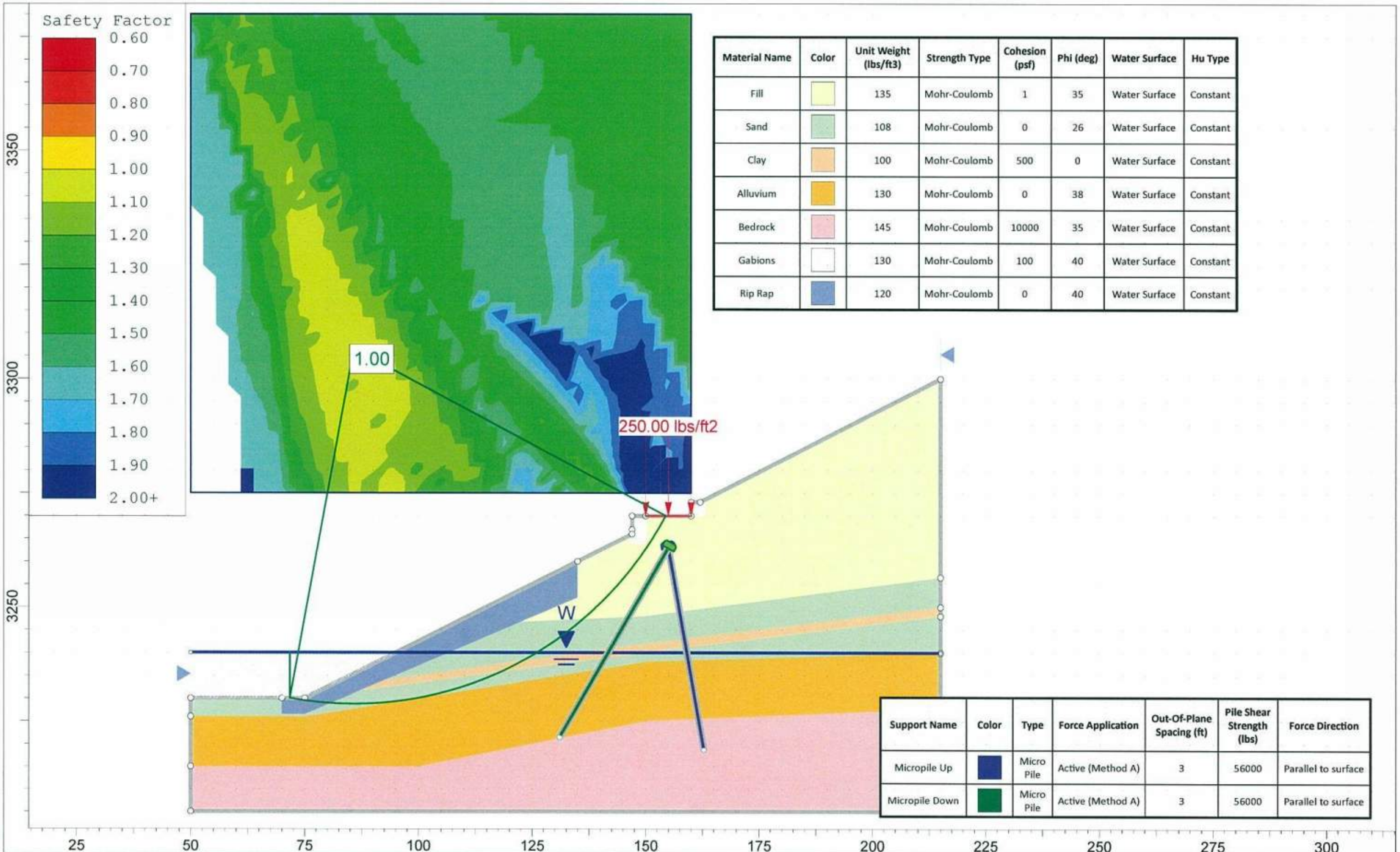
SLIDEINTERPRET 6.015

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, with Trail, Circular Analysis		
Drawn By JRD	*Scale* 1:350	*Company* Tt	
Date 2/20/2012		*File Name* I-90 slope-2mct.slim	

Figure 21



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Fill		135	Mohr-Coulomb	1	35	Water Surface	Constant
Sand		108	Mohr-Coulomb	0	26	Water Surface	Constant
Clay		100	Mohr-Coulomb	500	0	Water Surface	Constant
Alluvium		130	Mohr-Coulomb	0	38	Water Surface	Constant
Bedrock		145	Mohr-Coulomb	10000	35	Water Surface	Constant
Gabions		130	Mohr-Coulomb	100	40	Water Surface	Constant
Rip Rap		120	Mohr-Coulomb	0	40	Water Surface	Constant

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)	Force Direction
Micropile Up		Micro Pile	Active (Method A)	3	56000	Parallel to surface
Micropile Down		Micro Pile	Active (Method A)	3	56000	Parallel to surface

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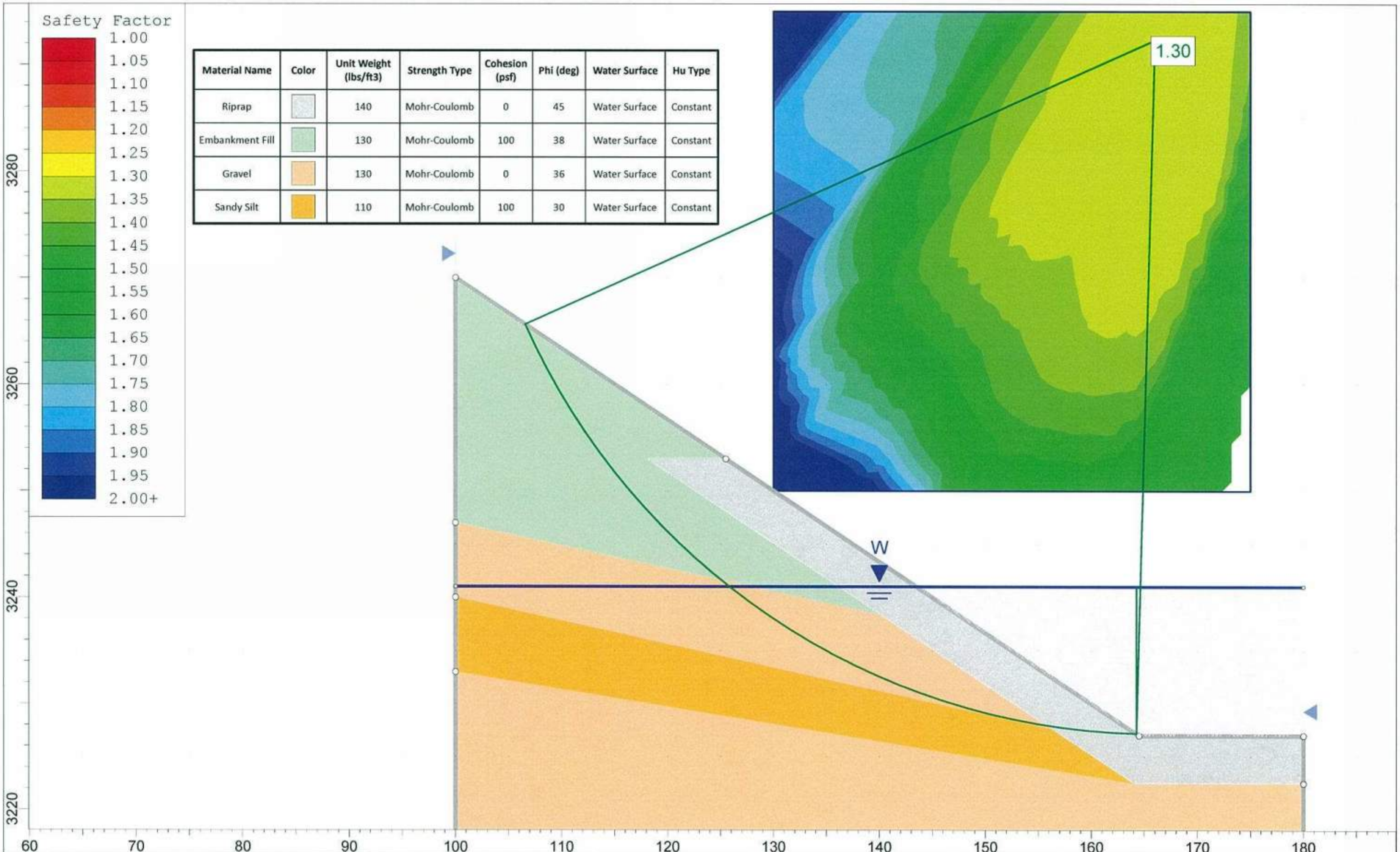
SLIDEINTERPRET 6.015

Project	Milltown Trails, I-90 Bridges West Slope		
Analysis Description	Existing 2H:1V Slope with Micropiles, Average Water Elev, with Trail and Traffic Loads, Circular Analysis		
Drawn By JRD	*Scale* 1:350	*Company*	Tt
Date 2/20/2012		*File Name*	I-90 slope-2mc.slim

Figure 22

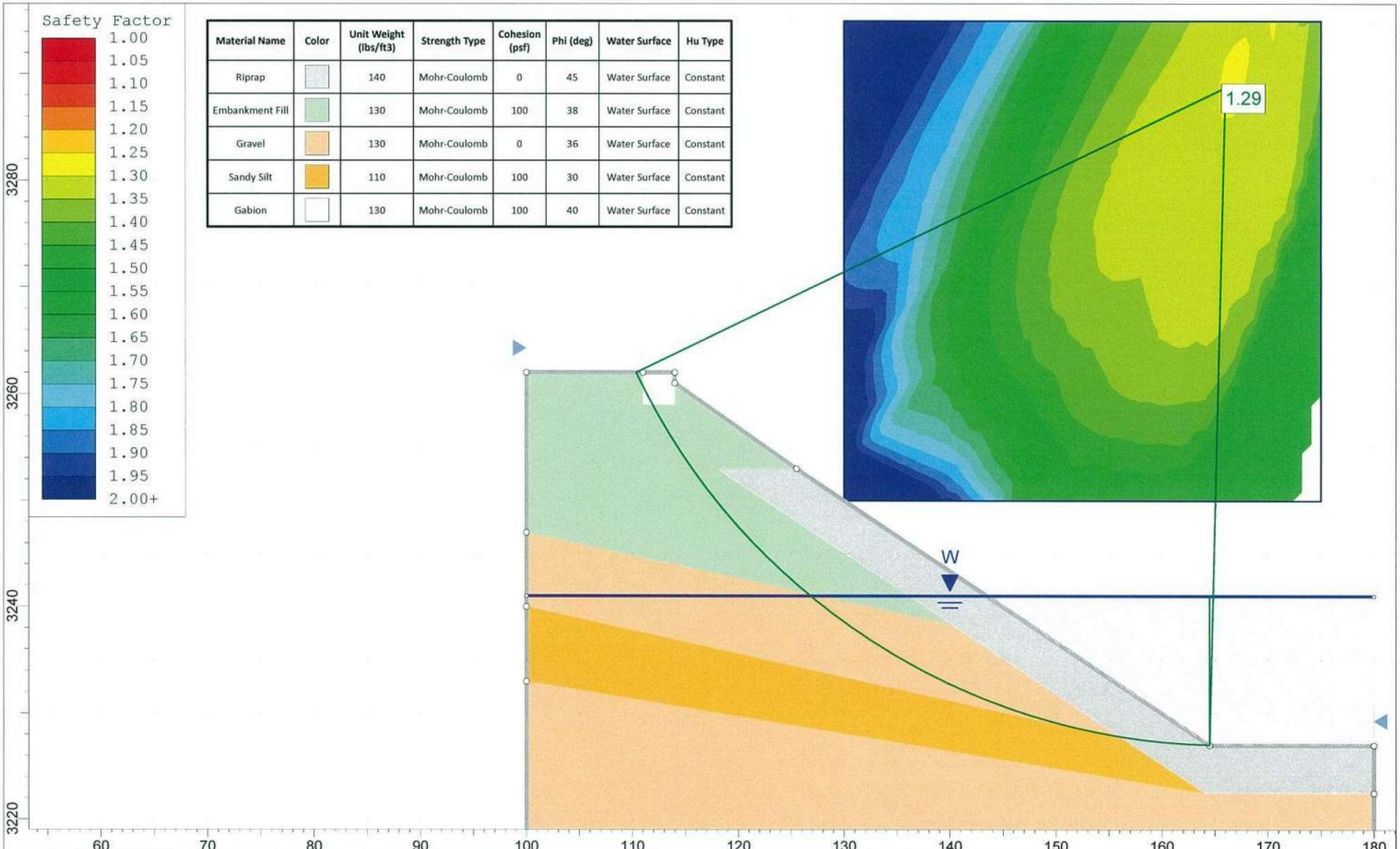


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Riprap		140	Mohr-Coulomb	0	45	Water Surface	Constant
Embankment Fill		130	Mohr-Coulomb	100	38	Water Surface	Constant
Gravel		130	Mohr-Coulomb	0	36	Water Surface	Constant
Sandy Silt		110	Mohr-Coulomb	100	30	Water Surface	Constant

SLIDEINTERPRET 6.015

Project	Milltown Trails, MRL Bridge West Slope		
Analysis Description	Existing 1.5H:1V Slope, Circular Analysis		
Drawn By: JRD	Scale: 1:150	Company	Tt
Date	2/21/2012, 9:19:06 AM	File Name	MRL slope 1.slim

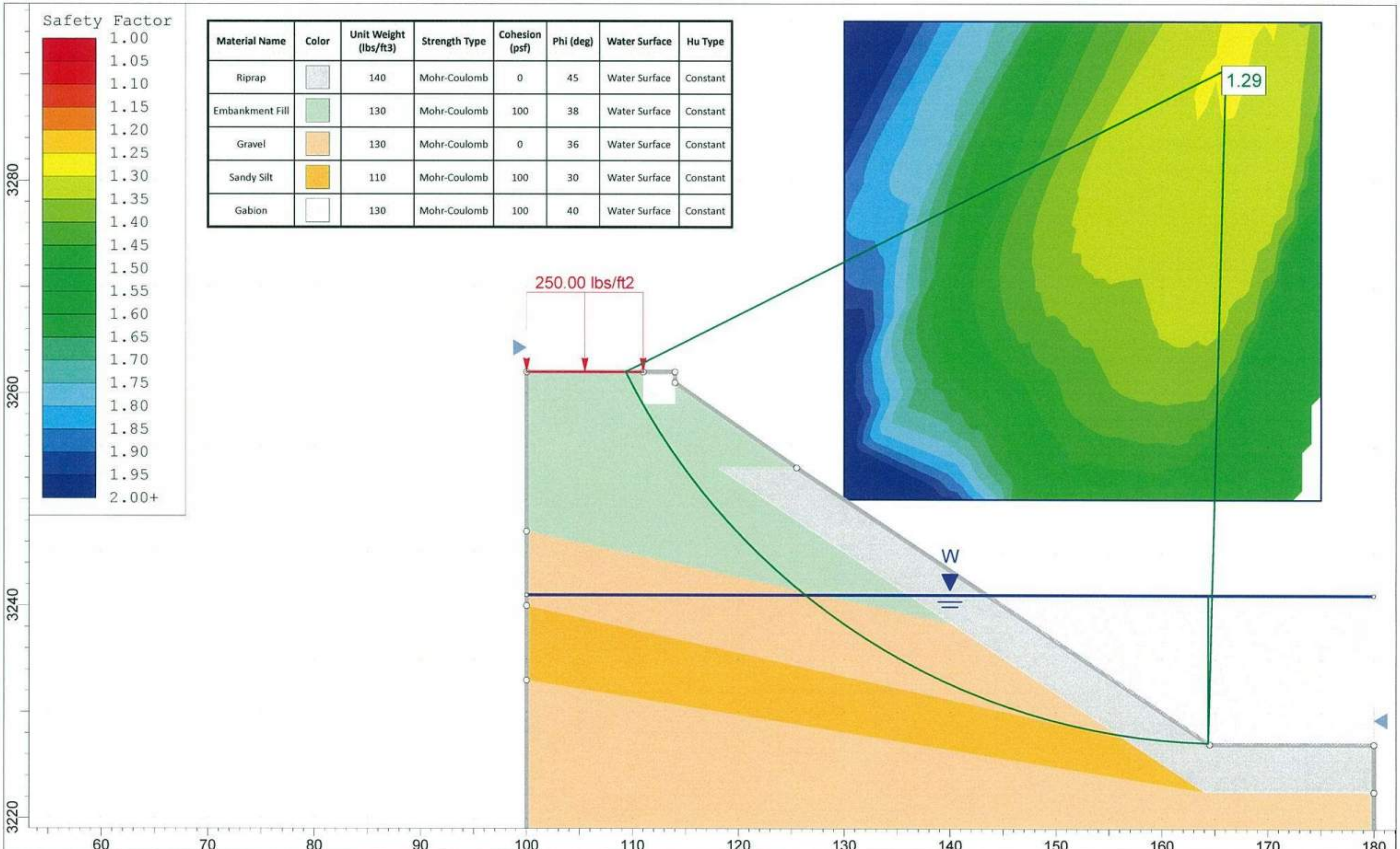
Figure 23



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Riprap		140	Mohr-Coulomb	0	45	Water Surface	Constant
Embankment Fill		130	Mohr-Coulomb	100	38	Water Surface	Constant
Gravel		130	Mohr-Coulomb	0	36	Water Surface	Constant
Sandy Silt		110	Mohr-Coulomb	100	30	Water Surface	Constant
Gabion		130	Mohr-Coulomb	100	40	Water Surface	Constant

rocscience	*Project*	Milltown Trails, MRL Bridge West Slope	
	Analysis Description	Existing 1.5H:1V Slope, With Trail, Circular Analysis	
	Drawn By JRD	*Scale* 1:150	*Company* Tt
SLIDEINTERPRET 6.015	*Date* 2/21/2012, 9:19:06 AM		*File Name* MRL slope 2.slim

Figure 24



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type
Riprap		140	Mohr-Coulomb	0	45	Water Surface	Constant
Embankment Fill		130	Mohr-Coulomb	100	38	Water Surface	Constant
Gravel		130	Mohr-Coulomb	0	36	Water Surface	Constant
Sandy Silt		110	Mohr-Coulomb	100	30	Water Surface	Constant
Gabion		130	Mohr-Coulomb	100	40	Water Surface	Constant

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SLIDEINTERPRET 6.015

Project	Milltown Trails, MRL Bridge West Slope		
Analysis Description	Existing 1.5H:1V Slope, With Trail and Traffic Load, Circular Analysis		
Drawn By	JRD	Scale	1:150
Company	Tt		
Date	2/21/2012, 9:19:06 AM	File Name	MRL slope 2.slim

Figure 25